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Volume III · EARTH, ROCK-FILL, STEEL AND TIMBER DAMS

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New York

JOHN WILEY & SONS, INC.

CHAPMAN & HALL, LTD.

London

fifth printing, april, 1955

PRINTED IN THE UNITED STATES OF AMERICA

CONTENTS

In this book article headings are listed in the table of contents only in the volume in which they occur, but all chapter titles for the three volumes are listed in each volume. A complete index covering all three volumes appears in each volume.

Volume I

Volume II

Volume III

CHAPTER 16. SOIL TESTS AND THEIR UTILIZATION

CHAPTER 17. EARTH DAMS—GENERAL PRINCIPLES OF DESIGN

CHAPTER 18. STABILITY OF EARTH DAMS

CHAPTER 19. DETAILS OF EARTH DAMS

CHAPTER 16[1]

SOIL TESTS AND THEIR UTILIZATION

1. General. The quality of the materials—ledge rock, rock, and soils—at and near the dam site is of the utmost importance. Whether the dam is to be of concrete or earth, most of the materials for its construction will usually come from points not far distant from the site.

The various tests which it is desirable to make on materials are listed and discussed. Except in special cases test procedure has not been given because such procedure is readily available elsewhere. Many of the desirable tests have been standardized by the American Society for Testing Materials and may be found in their manuals. Many of the desirable tests on soils have not been standardized. For suitable procedure for such tests the reader is referred to *Notes on Soil Testing for Engineering Purposes*, by A. Casagrande and R. E. Fadum, of the Graduate School of Engineering, Harvard University.

The significance of the tests made in connection with dam foundations and the materials utilized in dams is emphasized by numerical examples in some cases.

2. Terms Used in Soil Mechanics. Chapter 1 (Investigation of Dam Sites) includes material dealing with the sampling of rocks and soils.

It does not appear necessary here to go into the geologic definitions of the various kinds of ledge rock, but because the terms which are in general use in soil mechanics are less well known it seems desirable to give definitions of a few of the more important ones. Others will be defined later as used. Reference is also made to "Soil Mechanics Nomenclature," American Society of Civil Engineers Manuals of Engineering Practice No. 22, which contains a rather complete set of definitions and glossary of terms on soil mechanics. An attempt has been made to follow that nomenclature wherever applicable. In the following paragraphs, the symbols adopted in A.S.C.E. Manual 22 are given in parentheses.

Cohesion per unit area (c). This is the no-load unit shear strength of a given material. On the usual shear diagram, it is the vertical intercept at zero normal load. It is usually stated in terms of tons or pounds per square foot.

Angle of internal friction (ϕ). If on a shear strength diagram (Fig. 10) where normal load is plotted as abscissas and shearing stress as ordinates a line is drawn connecting the yield or failure points, the angle which this line makes with the horizontal is considered to be the angle of internal friction of the given material.

[1] The nomenclature utilized in Chapters 16 to 19 agrees in so far as practicable with the Manual of Engineering Practice, No. 22, of the American Society of Civil Engineers, on "Soil Mechanics Nomenclature."

Shear strength (s). Shear strength is generally considered to be made up of two components—the no load shear strength or cohesion, as above, and that portion of shear strength which varies directly with the normal load. The usual equation of unit shear strength of a soil is

$$s = c + p \tan \phi \qquad [1]$$

where s = total unit shear strength,
c = cohesion or no load shear strength,
p = normal load,
ϕ = angle of internal friction for the material under the condition tested.

Units are usually in tons per square foot. For sands and gravels the value of c is usually zero, but for clays and ledge rock it may be a considerable part of the total shear strength.

The concept and definition represented by the above equation is disputed by some soil mechanicians as not being substantiated by precise tests, and for certain clays it has been established that both c and ϕ may vary with different loads. However, it is believed to be sufficiently close to the truth throughout a large range of materials to be a very useful concept.

Effective size (D_{10}). This is the grain size on a mechanical analysis curve (Fig. 2) than which 10 per cent is smaller and 90 per cent is coarser. This is the effective size as utilized by the late Allen Hazen.

Uniformity coefficient (C_u), also as used by the late Allen Hazen, is the ratio of that size than which 60 per cent is finer to that size than which 10 per cent is finer.

$$C_u = \frac{D_{60}}{D_{10}}$$

Void ratio (e). The ratio of the volume of voids to the volume of solids is

$$e = \frac{n}{1 - n}$$

Porosity or per cent of voids (n). The ratio of the voids to the total volume of the soil mass is

$$n = \frac{e}{1 + e}$$

Compaction, degree of (D_d), or degree of density, is defined by the equation

$$D_d = \frac{e \text{ (in sample)} - e \text{ (in densest state)}}{e \text{ (in loosest state)} - e \text{ (in densest state)}}$$

Plastic limit (w_p) is the lower limit of the plastic state, expressed as the minimum water content at which a soil can be rolled into a thread ⅛ in. in diameter without crumbling.

Liquid limit (w_l). The upper limit of the plastic state. It is the moisture content at which a trapezoidal groove of specified dimensions cut through a sample of soil will be just closed by 25 shocks or blows produced in a standard liquid limit device.

Specific gravity (G). The ratio of the weight in air of a material to the weight in air of an equal volume of distilled water all taken at a given temperature.

Dry density. The weight per unit of volume of the material after the elimination of all moisture. Thus a cubic foot sample of soil obtained from a test pit may weigh 120 lb but after it is thoroughly dried and all moisture is eliminated it may weight only 90 lb. The latter figure is said to be the dry density per cubic foot.

Moisture content or water content (w). The ratio of the weight of water in a given soil mass to the weight of solid particles.

Optimum water content (w_0). The water content at which the maximum density is produced in a soil by a specific amount of compaction.

Degree of saturation (S). The ratio expressed as a percentage of the volume of water in a given soil mass to the total volume of intergranular soil space (voids). Thus $S = V_w/V_v \times 100$.

Coefficient of permeability, also referred to as transmission constant (k), is the quantity of water flowing through a unit area of soil at unity hydraulic gradient in unity time.

3. Classification of Soils. For engineering purposes it is most convenient to classify soils by sizes, because the size of the particles is the most important factor in determining its physical behavior from an engineering standpoint. The mineralogical character of the particles, however, also has a bearing on the qualities of a soil.

The method of determining the grading of a soil (mechanical analysis) is briefly as follows:

After a thorough drying in the oven, a sample of the material is passed through a series of standard sieves. The shaking is done either by hand or by use of a rotap machine. For that portion of the sample which passes the 200 mesh sieve, the analysis is completed by use of the hydrometer method. This method is dependent on the fact that for any given material the finer the size of particles the slower they will settle through the water.

Thus the density of a mixture of fine soil particles and water may be determined by a hydrometer and the amount of solids present indicated. If the density of the mixture is obtained at frequent intervals, the percentage of the various-sized particles present may thus be determined indirectly.

Fig. 1 gives several of the usual bases for classifying soils. Of the three bases given, that of the Bureau of Soils of the United States Department of Agriculture appears to be the more generally utilized. Accordingly, herein, unless otherwise mentioned it will be understood that classification is according to the U. S. Bureau of Soils. The Massachusetts Institute of Technology classification, however, has an advantage in that the finer materials are further subdivided.

In Fig. 2 is given several mechanical analyses of different kinds of soils. It does not seem advisable to attempt to be too rigid in classifying soils in accord-

ance with their mechanical analysis. Thus a certain material might have only 25 or 30 per cent of clay (material finer than 0.005 mm), but the material as a whole might exhibit most of the qualities of clay. The classifications appearing on the analyses curves are merely intended to be descriptive.

Colloidal material. The fat clay in Fig. 2 probably has a considerable percentage of material finer than 0.0002 mm, which is classified as colloidal. The

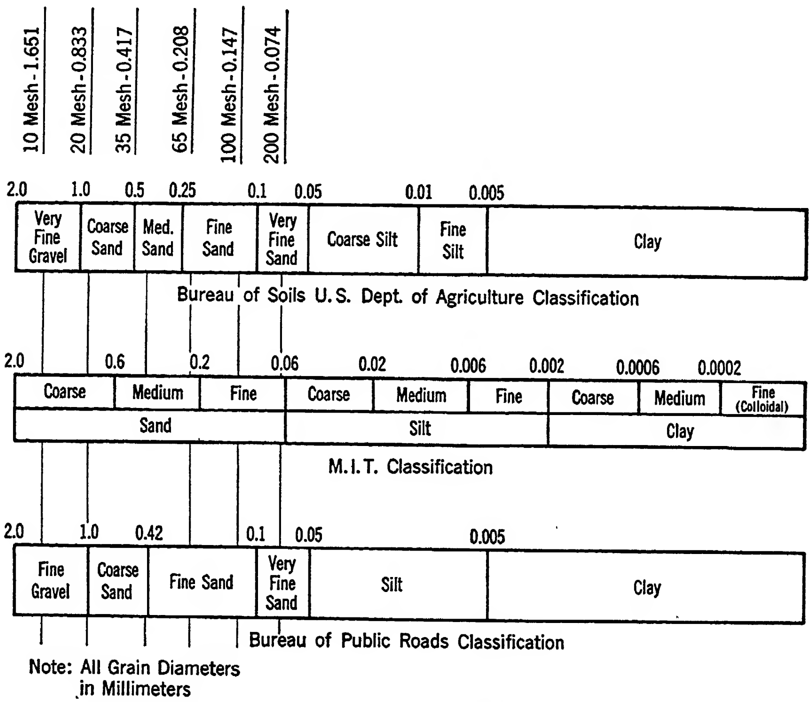

FIG. 1. Soil classifications in general use.

presence of colloids in appreciable percentages may restrict the field of usefulness of the material. In water colloidal material appears not to obey the laws of gravity. The colloidal particles float around indefinitely in accordance with the theory of "Brownian" movement. Viewed through a powerful microscope, the colloidal particles look gelatinous or amorphous.

The capacity of clays high in colloids for taking up large quantities of water makes colloids of particular interest in the design and construction of earth dams. Thus some clays, when sluiced in hydraulic or semi-hydraulic fill dams, will expand from 1.5 to 3.5 times their volume (as determined when thoroughly dried at a temperature of 212 to 230° F). A very large portion of this water is retained almost indefinitely and causes the mass to remain in an almost liquid or semi-liquid condition sometimes for many years. There is very little tendency for the

water to drain out of the material; consolidation, which is very slow, results almost entirely from pressure.

There is a marked difference in the plasticity of some materials having particles of equal fineness. Thus a quartz sand composed of excessively fine particles ("rock flour") will show practically no plasticity, whereas a colloidal clay of equal fineness will have great plasticity. Rock flour having an effective size as low as 0.005 mm may usually be successfully used in the core of earth dams built by the hydraulic or semi-hydraulic fill method. Such material will settle out of the

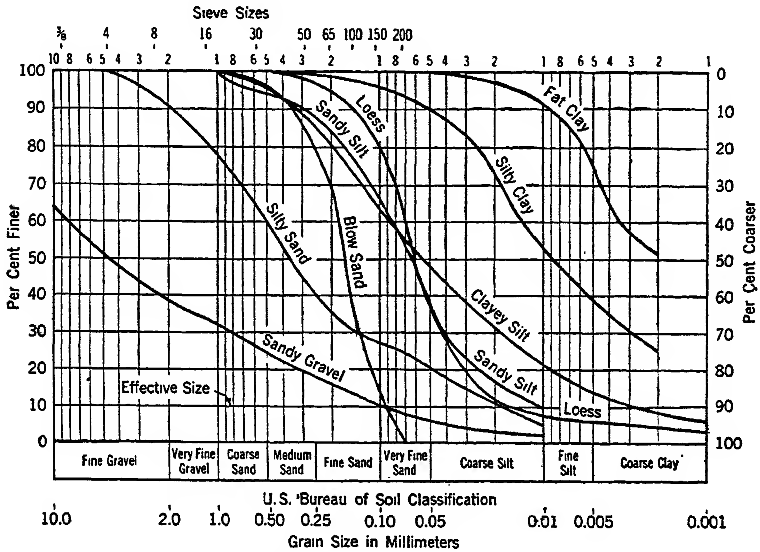

FIG. 2. Mechanical analysis of various soils.

water in the central pool quite readily and will reach a stable condition in a comparatively short time. A clay high in colloids, on the other hand, when hydraulicked, swells 1.5 to 3.5 times its dry volume and remains in a semi-liquid condition almost indefinitely. Its use in high percentages is seldom advisable in a hydraulic or semi-hydraulic earth dam unless heavy upstream and downstream fills of stable material and sufficient dimensions are included in the design to retain it.

The fat clay shown in Fig. 2 has 74 per cent finer than the limiting upper clay size of 0.005 mm, and all the rest of the material is a silt.

The silty clay shown in Fig. 2 has 39 per cent finer than the limiting upper clay size of 0.005 mm, and except for about 10 per cent of sand all the rest of the material is silt.

The clayey silt shown in Fig. 2 has 14 per cent of clay, 34 per cent of silt, and most of the rest of the material is fine sand.

The sandy silt shown in Fig. 2 has about 40 per cent finer than the limiting upper silt size of 0.05 mm, and most of the rest of the material is fine sand.

The silty sand shown in Fig. 2 has only 20 per cent finer than the limiting upper size of silt (0.05 mm); 22 per cent of the sample would be classed as fine gravel under the Bureau of Soils Classification, and the remainder is sand.

The sandy gravel shown in Fig. 2 has 68 per cent coarser than the limiting lower size of fine gravel, and practically all of the remainder of the material is sand.

The loess shown in Fig. 2 is a relatively fine wind-blown and wind-deposited soil which is quite common in the Great Plains section of the country west of the Mississippi River. To a lesser extent it also occurs east of the Mississippi River, particularly in the Southern States. Its principal characteristic is its uniformity in size. Thus only 20 per cent is coarser than 0.1 mm and only 11 per cent finer than 0.02 mm. Almost 40 per cent of it is finer than the limiting upper silt size of 0.05 mm.

The blow sand shown in Fig. 2 has the same wind-blown origin as loess and is also extremely uniform in size, but it differs from loess in being much coarser. Thus the sample of blow sand, the analysis for which is given in Fig. 2, is coarser than the limiting upper size for silt of 0.05 mm and is all in the sand sizes.

It will be noted that in naming the materials for which mechanical analyses are given in Fig. 2, in some cases the material is called a clay or a silt when there is less than 50 per cent of that material in it. This is because the finer material (although less than 50 per cent of total) often provides the dominating characteristics. Thus the silty clay given above has only 39 per cent finer than the clay size of 0.005 mm.

In all field investigations, the designation of samples of soil in the field is always troublesome, owing to the difference in judgment of various engineers and inspectors. As a result, many field designations mean very little. Consequently some effort is usually made to delimit the range in which judgment may be utilized. It is evident that if a definite way of numbering various gradings of soils is adopted, the classifying could be more exact, and any material falling into a given classification would possess certain quite definite characteristics and qualities. Also uniformity of classification could be secured because each field inspector and engineer engaged in classifying soils would have a set of synthetic samples in glass bottles for comparison with the natural material from the bore hole and would classify accordingly. Such a scheme, named "The Kendorco Classification" from the engineers who originated it,[2] was utilized in connection with the subsurface investigations at the Quabbin Dam and Dike in Massachusetts. The "Modified Kendorco Classification" here presented in Fig. 3 is the work of W. I. Kenerson, soils engineer, and Frank E. Fahlquist, geologist, of the Providence District of the Army Engineers. It has been utilized in connection with the investigation of more than 20 dam sites in New England.

Under this system of classification soils are divided into 13 classes of material instead of 9, as in the original Kendorco system. The even numbers in the classification are used to designate materials of relatively uniform grading and odd numbers to designate materials showing considerable grain size variation.

[2] See STANLEY M. DORE, "Permeability Determinations, Quabbin Dams," *Trans. Am. Soc. Civil Engrs.*, Vol. 102, 1937, p. 682.

Thus the even-numbered mechanical analysis curves in Fig. 3 are steep and the odd-numbered curves have a relatively flat slope. Fig. 3 shows the limits of soil classes in the Modified Kendorco Classification and Table 1 gives a description of the classes. The authors believe that one of the greatest advantages of the scheme is that with a little practice it is possible to classify a soil by visually comparing it with a set of standard samples.

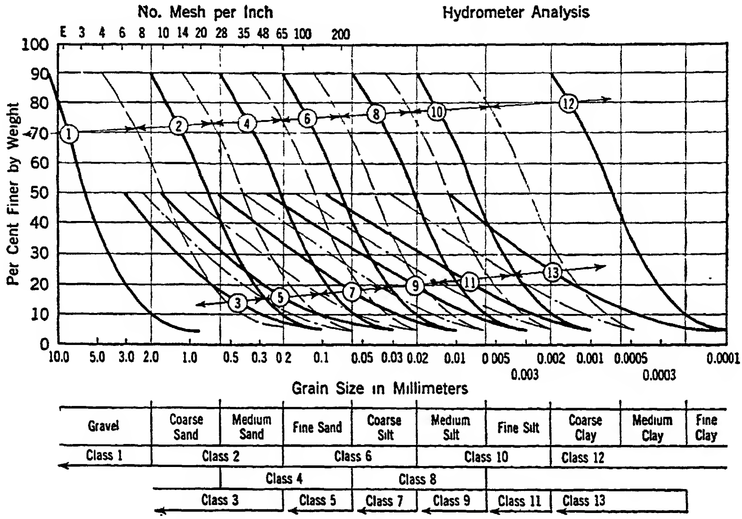

FIG. 3. Modified Kendorco Classification, limit of soil classes.

TABLE 1

MODIFIED KENDORCO CLASSIFICATION OF SOILS

Class	Description of Material
1	*Relatively clean gravel.* A material the bulk of which consists of gravel. Contains a small per cent of coarse sand but very little, if any, fine sand.
2	*Uniform coarse to medium sand.* A material the bulk of which consists of coarse to medium sand. Contains little gravel and fine sand.
3	*Variable gravel and coarse to medium sand.* A mixed material of which the controlling part is coarse to medium sand. Contains much gravel and little fine sand.
4	*Uniform medium to fine sand.* A material the bulk of which consists of medium to fine sand. Contains a small per cent of grains larger than medium sand and smaller than fine sand.
5	*Variable medium to fine sand.* A mixed material of which the controlling part is medium to fine sand. Contains much coarse sand and gravel and little fine silt.
6	*Uniform fine sand to coarse silt.* A material the bulk of which consists of fine sand to coarse silt. Contains a small per cent of grains larger than fine sand and smaller than coarse silt.

TABLE 1—*Continued*

MODIFIED KENDORCO CLASSIFICATION OF SOILS

Class	*Description of Material*
7	*Variable fine sand to coarse silt.* A mixed material of which the controlling part is fine sand to coarse silt. Contains much coarser sand and gravel and little fine silt.
8	*Uniform coarse to medium silt.* A material the bulk of which consists of coarse to medium silt. Contains small per cent of grains larger than coarse silt and smaller than medium silt.
9	*Variable coarse to medium silt.* A mixed material of which the controlling part is coarse to medium silt. Contains much sand, little gravel and fine silt.
10	*Uniform medium to fine silt.* A material the bulk of which consists of medium to fine silt. Contains a small per cent of clay and grains larger than medium silt.
11	*Variable medium to fine silt.* A mixed material of which the controlling part is medium to fine silt. Contains much coarser silt, sand, and possibly gravel and little clay.
12	*Uniform clay.* A material the bulk of which consists of particles ranging from coarse clay to colloidal sizes. Contains a small per cent of silt grains.
13	*Variable clay.* A material composed of clay and much coarser material ranging in size from silt to coarse sand. Possesses characteristics of clay.

4. Field Laboratory. Some sort of field laboratory is desirable at or near the job during the period when investigations are being made. Whether it is decided to build the dam of earth or concrete, the services of a laboratory will be required both during the preliminary period and also for control purposes during construction. The decision as to whether to establish one during the preliminary period will often be determined by nearness to the work of an existing suitably equipped laboratory.

A completely equipped laboratory for making soil tests, tests on concrete aggregate, and tests on concrete and rock may be a necessity on a large job. Cost of equipment will range from $2,000 to, say, $15,000, and the personnel required in sampling and testing will range from 3 or 4 men to 30 or 40, according to the size of the job and the problems involved.

Not all of the personnel need be trained and experienced men. On work of magnitude, the testing and sampling work is usually divided into two divisions with a soils engineer in charge of one branch and a concrete technician in charge of the other.

On a job of moderate size, it is usually advisable to obtain the services of an existing well-equipped laboratory and to have on the job only the necessary equipment for sampling and making the simpler tests. Even in this case, however, it is essential to have on the job a trained soils engineer for supervising the sampling operations because even the most precise testing of the best laboratory may be absolutely meaningless if sampling is ignorantly done.

A soils laboratory which might be suitable in connection with the investigation of several dam sites of moderate size might include the following equipment.

1. Mechanical analysis sieves, together with a rotap machine for shaking them.
2. Soil hydrometers and flasks for determining the percentages of the sizes finer than can be obtained by sieves.
3. Balances for weighing small fractions of sample.
4. Scales for weighing materials.
5. Electric shakers such as are used in soda water fountains for hydrating the portion of samples on which hydrometers are to be used.
6. Electric drying oven.
7. Flasks for specific gravity determination.
8. Permeameters for determining transmission constants of various materials.
9. Consolidometers for making consolidation tests.
10. Horizontal shear machine for making shear tests on materials (silts, clays, and sands) as in Figs. 4 and 5.
11. Equipment for determining tannic acid contents of sands and gravels for concrete.
12. Electric refrigeration for making freezing and thawing tests of coarse aggregates for concrete.
13. Dial extensometers.
14. The usual trays, pans, jars, etc., without which any laboratory would be incomplete.

5. Tests of Soils Consisting Mainly of Coarse Sands and Gravels. For coarse sands and gravels tests should, in general, be made to determine the following properties unless the answers are already known within sufficiently close limits for the purpose in hand. The designations in parentheses refer to test procedure specifications of the American Society for Testing Materials.

1. Mechanical analyses (E11–26) (C41–36) (C33–37T).
2. Permeability (determination of transmission constant or permeability coefficient unless permeability is known within sufficient limits from (1).
3. Void ratio.
4. Presence of soluble material.
5. Specific gravity of particles (C127–36T) (C128–36T).
6. Dry density (dry weight per cubic foot) (C29–27).
7. Direct shear tests not always necessary with clean coarse sands and gravels, as shear strength is usually high and such materials are known to possess an angle of internal friction in excess of 30°.

6. Tests of Soils Consisting of Clays, Silts, and Fine Sands. For clays, silts and fine sands, the following tests utilizing undisturbed samples are applicable. Designations in parentheses refer to test procedure specifications of the American Society for Testing Materials. Laboratory procedure in the testing of soils for engineering purposes has not yet been fully standardized. One of the best brief summaries of approved laboratory procedure for the tests mentioned below is given in *Notes on Soil Testing for Engineering Purposes* by A. Casagrande and R. E. Fadum, Soil Mechanics Series No. 8, published by the Graduate School of Engineering, Harvard University.

1. Mechanical analysis (D422–39).
2. Hydrometer analysis of fraction passing finest sieve in above.[3]
3. Permeability tests (not necessary in many cases).
4. Void ratio.
5. Moisture content.
6. Plasticity limit.[3]
7. Liquid limit.[3]
8. Specific gravity of particles with Le Chatilier flask (C77–40).
9. Density (dry weight per cubic foot).
10. Percentage of soluble materials.
11. Direct shear tests.
12. Triaxial shear tests (occasionally, particularly where critical density of very fine sands is in question).
13. Consolidation tests.
14. Optimum moisture content.
15. Expansion after consolidation.
16. Shrinkage limit.

7. Tests of Concrete Materials. Large samples of quarried rock should be broken down in a commercial crusher, as a laboratory crusher may not give the same grading or shape of particles. The designation in parentheses refers to Test Procedure Specifications of the American Society for Testing Materials, Philadelphia, Pa.

1. Mechanical analyses (E11–26), (C41–36).
2. Voids in concrete aggregate (C30–22).
3. Unit weight of aggregate (C29–27).
4. Freezing and thawing tests.
5. Soundness of aggregates by use of sodium sulphate or magnesium sulphate (C88–37T).
6. Organic impurities in sands for concrete (C40–33).
7. Abrasion of rock (D2–33) (C131–37T).
8. Specific gravity of particles (C127–36T) (C128–36T).
9. Percentage of soluble material.
10. Crushing strength of rock from which aggregates are made.
11. Crushing strength of concrete cylinders made from various mixtures of the aggregates and standard Portland cement (C31–33) (C39–33).
12. Structural strength of fine aggregates using constant water cement ratio mortar (C87–36).
13. Toughness of rock (D3–18).

8. Tests of Foundation Materials. If an earth dam is to be constructed on a soil foundation, the tests already given in Arts. 5 and 6 are applicable. If the foundation is ledge rock, the quality of the rock should be tested unless its quality is unquestionable. Often when ledge rock is once encountered and the weathered zone is removed the strength of the rock is so great in relation to that of the concrete which will be utilized in the structure that there is no point in learning just how great it is. With many ledge rock foundations, however, this is not true, so thorough tests are required.

[3] There is an American Society for Testing Materials procedure for this test, but it is not generally accepted by soils technicians.

This equation is not always strictly true in the case of some cohesive soils, but it is nevertheless a useful tool. For some fat clays the value of c and ϕ may vary between quite wide limits according to the conditions of loading.

In those shear machines which provide only uncontrolled strains, such as a weighted lever, instead of the proving ring, it is impossible to obtain the right-hand end of the curve below the peak. In such cases the actual ultimate strength cannot be determined, and recourse is had to noting the load when the left-hand rising portion of the curve exhibits a break or "knee," as at Y in Fig. 9. In some tests this knee is pronounced and is fairly close to the horizontal line corresponding to the ultimate strength, and it is assumed to be the ultimate strength.

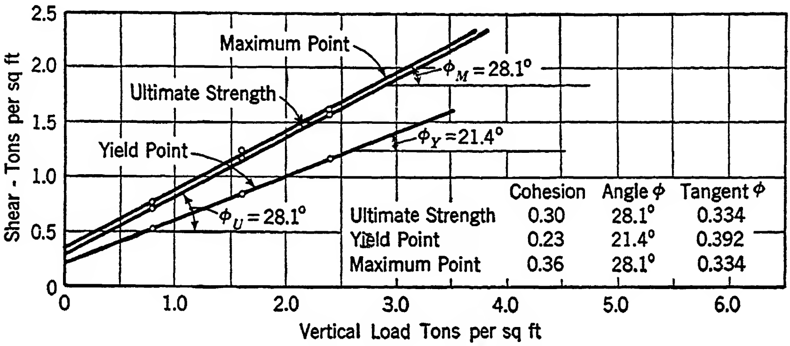

Fig. 10. Results of direct shear tests on a clayey sandy silt (same material as in Fig. 9).

In other cases, as in Fig. 9, it is not pronounced and only a wizard could pick out the alleged yield point. In Fig. 9 the so-called yield point is much below the ultimate strength.

It is, of course, better to use a type of machine which will give the true ultimate strength instead of the yield point. In any event all reports of shear tests should state which is used.

Note that, for this particular material, the ultimate strength is not materially less than the maximum point and considerably greater than the yield point. The difference between the maximum point and the ultimate strength may be large in some cases and approach zero in others.

Suitable procedure for shear tests and for other tests on soils which in many cases are of equal importance will be found in *Notes on Soil Testing for Engineering Purposes*, by A. Casagrande and R. E. Fadum, Soil Mechanics Series No. 8, published by the Graduate School of Engineering, Harvard University.

10. Consolidation and Its Significance. Consolidation as applied to soils means the process of becoming denser. When a soil is deposited from water or from the air, it is in a relatively loose state. As more and more material is deposited the soil first deposited becomes denser and denser as the superimposed load becomes greater and greater.

Under any given load a particular soil will finally reach a density such that it will not become any denser so long as the condition of loading remains the same.[5] The soil is then said to have reached a state of 100 per cent consolidation under that loading. Thus the soil foundation of an earth dam 100-ft high might have reached 100 per cent consolidation within, say, 1 year after the completion of construction, but if the height of the dam was then increased to 200 ft, the foundation would not be at 100 per cent consolidation until some time after the 200-ft dam was completed. In other words consolidation is for a certain loading.

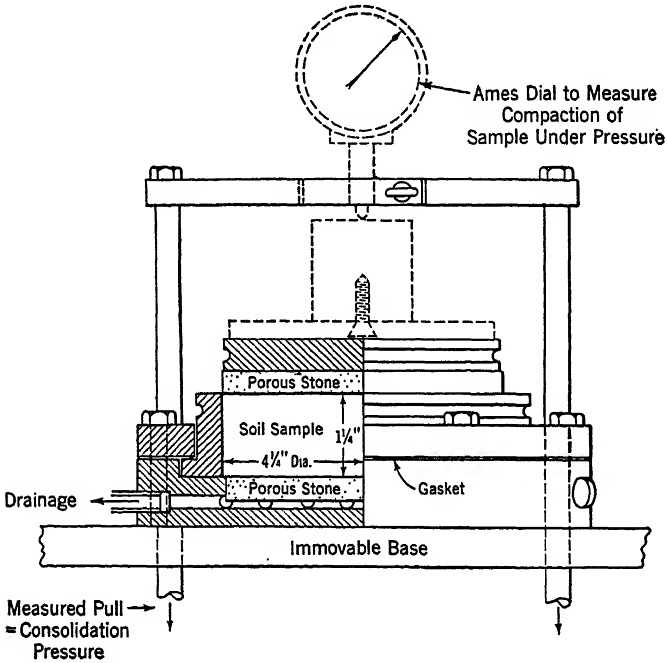

FIG. 11. Details of consolidation device as used at Zanesville, Ohio, Laboratory, U. S. Army Engineers.

There are cases, however, where the actual settlement due to the construction of a 200-ft dam is extremely slight because the soil had previously been consolidated by a tremendously great load ("preconsolidation loading") and had not greatly expanded on the release of that load.

For instance, the void ratio of the Trinity sands (Texas and Oklahoma), is sometimes as low as 0.20, which means a dry density of 140 lb per cu ft. Only the load of a very deep overburden could have caused even a very well graded sand to reach such a density. Consequently, no one would worry about settlement no matter how great the load of the dam placed upon it. The Trinity

[5] If a soil is vibrated as by an earthquake it may settle and become denser even though the amount of the superimposed load may not change at all.

sands would be at one end of the consolidation range while at the other end would be some of the very fat clays which have been subjected to very little load.

11. Void Ratio. The void ratio, which is the ratio of the volume of voids to the volume of solids, is used a very great deal and is determined in the laboratory as indicated by the following formula:

$$e = \frac{V(62.5G) - W}{W}$$

in which e = void ratio,
V = volume of the sample in cubic feet,
G = specific gravity of the grains,
W = weight of the sample (after drying) in pounds.

It is a simple matter to translate void ratio to porosity, which is the same as per cent voids. Thus

$$n = \frac{e}{1 + e}$$

in which n = porosity and e = void ratio.

12. Consolidation of Coarse Sands and Gravels. Coarse, well-graded sands and gravels, particularly if they are water deposited, reach a state close to 100 per cent consolidation very promptly after any given load is applied. Such materials seldom give concern to the engineer as far as consolidation is concerned.

13. Consolidation of Fine Sands. Fine sands will also consolidate very promptly under load, but even though they consolidate 100 per cent under the given load, they may not be in a very stable condition, particularly if the particles are relatively uniform in size. Casagrande has called attention to the critical condition of some such soils.

From his researches Casagrande determined that every cohesionless soil has a certain critical void ratio, void ratio at which it can undergo deformation without change of volume. The material is then said to be at its "critical density."

If a cohesionless soil is being deformed, as at failure in a direct shear machine, and it continues to expand in volume, it is below critical density (i.e., more dense than critical density). Materials that have a density greater than critical density are apt to be very stable, a condition which is usually found in nature in coarse, well-graded sands and gravels.

On the other hand, if a cohesionless soil contracts on deformation it is above critical density (i.e., less dense than critical density). With material in this condition, if saturated and under load, slight deformation will transfer part of the load to the water and a flow slide may occur. In nature fine uniform sands in a loose natural state may be very close to this condition. Consequently such materials, which have high void ratios (or low unit dry weights), should be regarded with suspicion and should be carefully investigated.

In Fig. 12 is shown the critical void ratios ("critical density") of a blow sand as determined from shear tests.

In any section of the country which is arid or semi-arid, the engineer should view the soils with suspicion insofar as they might be used for the foundation of a dam or for its embankment.

If a material that has never been really wet is to be utilized as the foundation of a dam under conditions such that it will always be saturated, its behavior should be thoroughly investigated. The dry density of a foundation material is at least some indication of the possibility of trouble. If one finds that the loess [6] foundation of an earth dam has a dry density of, say, 73 lb per cu ft, he should at least take warning and investigate the matter further.

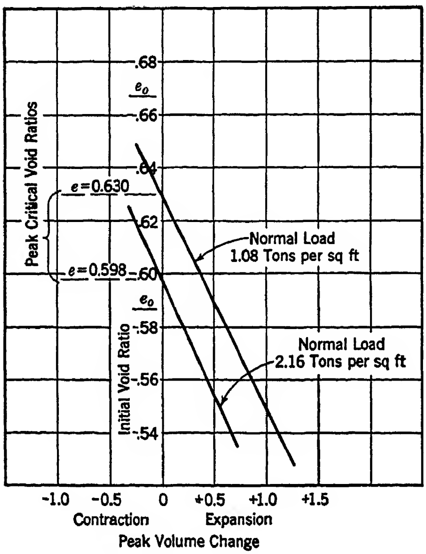

FIG. 12. Critical void ratios of a "blow sand" determined from shear tests.

An earth dam in an arid country was constructed of loess on a foundation of sandy loess. The loess in the embankment was moistened and properly compacted, and its dry density was 103 lb per cu ft. The foundation consisted of practically the same material but in its natural loose state had a dry density of only 73 to 80 lb per cu ft.

After the headwater was raised to about 50 per cent of the height originally intended, piping below the toe caused the cessation of reservoir filling and it became necessary to sink exploratory shafts and make other investigations.

This investigation revealed that the dam had settled irregularly. As soon as the rising reservoir level saturated the dry loose loess foundation, the loess contracted irregularly, leaving large voids under the embankment.

In other similar cases the presaturation of a loose, uniform, fine sand foundation has resulted in prompt consolidation.

At Franklin Falls, N. H., a loose fine and rather uniform sand foundation was consolidated by blasting so that a settlement of about 3 ft was induced in the foundation before the application of the load of the embankment.

14. Consolidation of Clays and Silts. In contradistinction to the consolidation of sands and gravels, the consolidation of clays and fine silts requires a much greater period of time and their rate and degree of consolidation may be a matter of grave concern to the engineer.

Assume that at the site of a proposed earth dam, the valley fill is composed of lenses of silty clay interspersed with deposits of sandy material. The material available for building the dam has been investigated and it has been found that

[6] Loess is a uniform fine sandy silt. See Fig. 2.

utilizing 1 on 3 slopes, the dam embankment itself will have an adequate factor of safety.

As so often happens, this leaves the foundation as the critical feature of the structure. Quick shear tests made on undisturbed samples from the silty clay lenses indicate that at present the foundation is not strong enough to safely support the load of the embankment. The question is, "Under the increasing load as the dam is constructed will the foundation consolidate enough to be safe both during construction and after the completion of construction?" It is this question that we should be able to answer by means of properly interpreted consolidation tests on undisturbed samples of the material.

15. Consolidation Device. There are several different forms of consolidation devices, but most of them are quite similar to that shown in Fig. 11. A measured downward pull on the yoke results in a predetermined unit compression on the soil sample. Porous plates at the top and bottom of the sample permit it to drain readily, and an Ames dial measures the contraction under any given load.

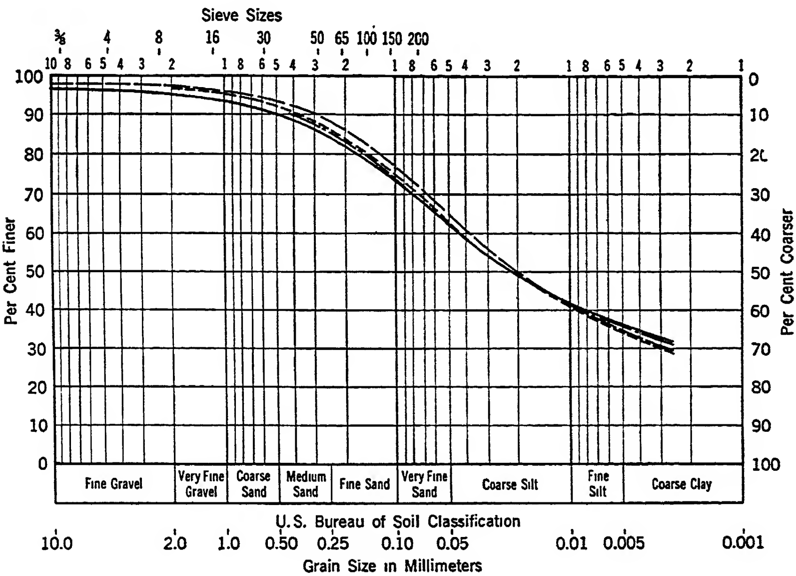

FIG. 13. Mechanical analysis of a sandy silty clay.

Note. Figs. 13 to 17 all deal with the same soil.

A given vertical load or pull is applied and the amount of contraction (or consolidation) is measured at intervals by reading the Ames dial. The void ratio, which is known at the start of the test, may be computed for any given movement (or consolidation) during the test. When the sample has reached full consolidation under the given load, a greater load is put on and the dial readings continued until complete consolidation is reached under the new load, and so on for as many different loadings as desired.

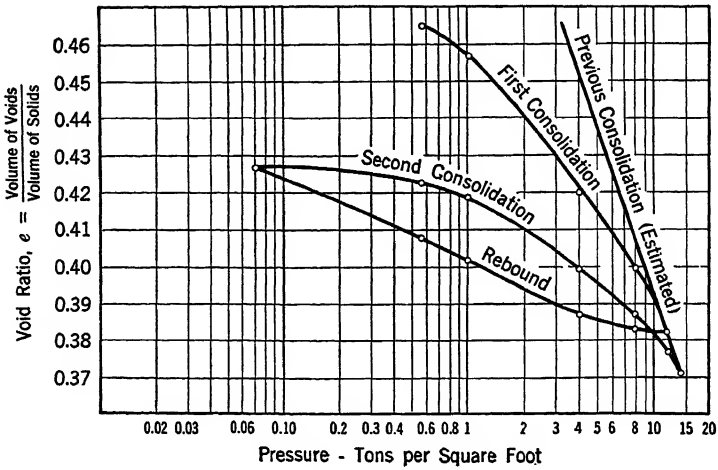

FIG. 14. Pressure-void ratio curve. Undisturbed sample saturated during test. Initial thickness of sample = 2.5 in. Cross-sectional area of sample = 24.80 sq in.

Note. Figs. 13 to 17 all deal with the same soil.

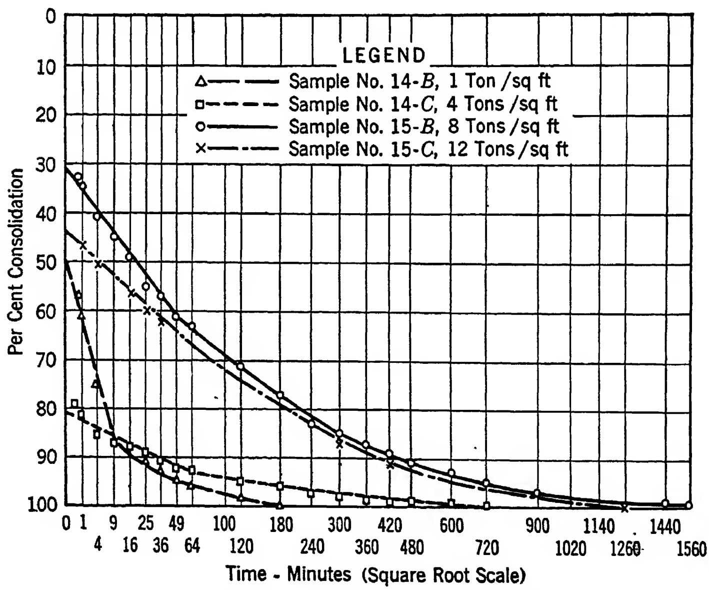

FIG. 15. Time-percent consolidation curves. Samples consolidated at 0.56 tons per sq ft before addition of final load. Consolidated samples used for consolidated shear tests. Undisturbed samples saturated during tests. Cross-sectional area = 24.80 sq in. Initial thickness = 2.5 in.

Note. Figs. 13 to 17 all deal with the same soil.

From the data obtained the void ratio versus per cent consolidation pressure curves may be plotted as in Fig. 14.

In Fig. 15 is given the time consolidation curves, and Fig. 13 gives the mechanical analysis of the material. It should be noted that the curves in Fig. 13 to 17 are for the same material, a silty clay about 30 per cent of which is clay finer than 0.004 mm.

16. Effect of Consolidation on Shear Strength. It now becomes necessary to know the effect of consolidation on the shear strength of the material, and at this

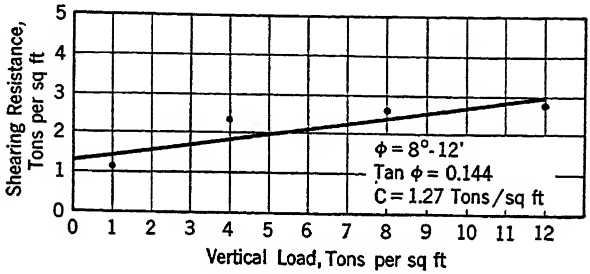

FIG. 16. Quick shear strength curves.

Note. Figs. 13 to 17 all deal with the same soil.

point it is easy to go astray in interpreting the results. A good way to interpret the results is to plot curves giving the shear strength of the material for various percentages of consolidation based on the various tests under different vertical loads. To do this, consolidated quick shear tests and quick shear tests should both be taken for the same material. The quick shear test shows the strength

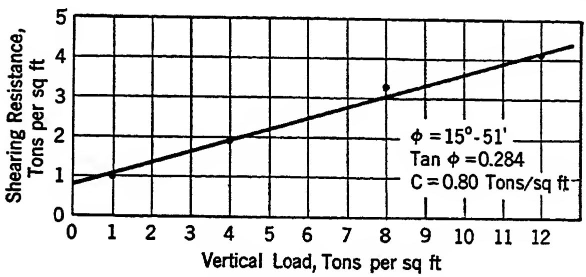

FIG. 17. Consolidated shear strength curves.

Note. Figs. 13 to 17 all deal with the same soil.

of the material now and the consolidated shear test the strength as it will be when fully consolidated under the given load.

Fig. 16 shows the results of such quick shear tests and Fig. 17 the results of consolidated quick shear tests.

Before we can compute from the above data the shear strength at various percentages of full consolidation we must determine the time required for consolidation in the prototype.

17. Time of Consolidation. The time required for the given material to reach various percentages of consolidation in the thin test specimens (2.5 in. thick in this case) drained top and bottom, under several different conditions of unit loading, is shown in Fig. 15. Thus in the test a loading of 12 tons per sq ft (equivalent to that of a dam, say, 200 ft high) will result in the specimen being 90 per cent consolidated within 400 min. Terzaghi showed that, for given unit loading, the time of consolidation varied as the square of the thickness of the layer. Thus

$$t = \frac{Td^2}{D^2} \qquad [2]$$

where t = time of consolidation in model,
d = thickness of layer in model,
D = thickness of layer in prototype,
T = effective time of consolidation in prototype, which in the case of steady construction is equal to one-half the elapsed time.

Example. What is the time period in laboratory test corresponding to a 4-month period of consolidation in the prototype for a plastic layer 18.5 ft thick, having, just above and below the plastic clay, layers of material which, as compared with the plastic clay, are tremendously pervious. The latter condition is simulated by the drainage conditions produced by the two porous stones used in the test (Fig. 11). A 4-month period of consolidation, assuming construction at an even rate, would be equivalent to an effective time of consolidation of 2 months.

$$T = \frac{4 \times 30 \times 24 \times 60}{2} = 86{,}500 \text{ min}$$

d, thickness of the laboratory consolidation specimen, is 2.50 in.

$$t = \frac{86{,}500 \times 2.5^2}{18.5^2 \times 12^2} = 10.95 \text{ min, or } 11 \text{ min}$$

Should the prototype be drained at the top, instead of the top and bottom, use one-half the thickness of the model layer or $d = 1.25$ in.

Referring to the time per cent consolidation curves in Fig. 15, it is found that for a normal load of 8 tons per sq ft and a time of 11.0 min, the per cent consolidation reached would be 46 per cent, which is also the per cent consolidation that would be reached in 4 months in the prototype.

Similarly for a 12-ton normal load, it is found that the per cent consolidation in the prototype would be 55 per cent in 4 months (or 11.0 min in laboratory test).

18. Settlement. It is frequently desirable to compute the amount of settlement which will result from a certain amount of consolidation. Per cent change in volume due to consolidation is

$$\frac{e_1 - e_2}{1 + e_1} \qquad [3]$$

and

$$S = \frac{(e_1 - e_2)}{(1 + e_1)} h \qquad [4]$$

where S = settlement in feet,
h = thickness of layer in feet,
e_1 = void ratio before consolidation,
e_2 = void ratio after consolidation.

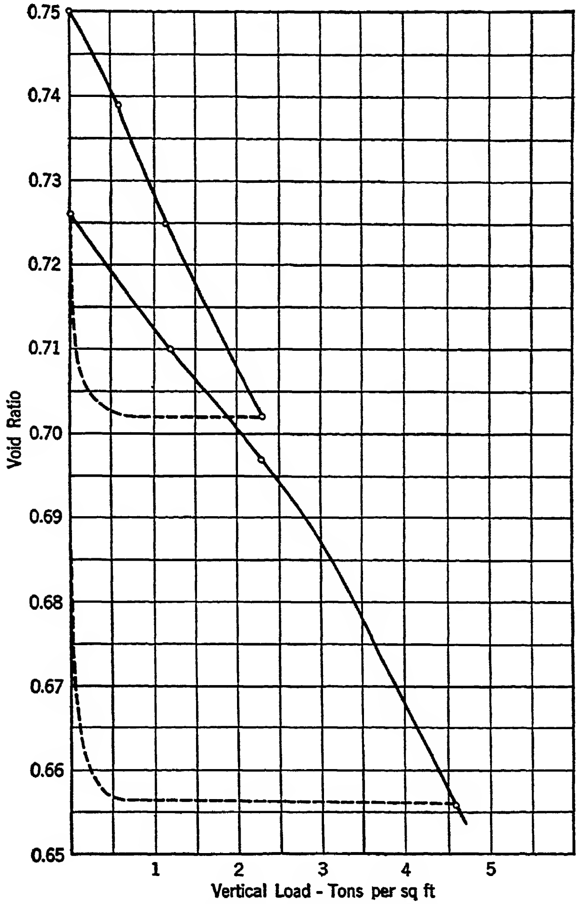

FIG. 18. Consolidation void ratio curve for a silty clay.

Example. In an earth dam being constructed on a somewhat yielding foundation it is desired to know what surcharge should be placed on top of the dam to compensate for the settlement. The dam is 70 ft high and the normal load at center line of the dam is 4 tons per sq ft.

Fig. 18 gives results of a consolidation test on the plastic stratum in the foundation which has a thickness of 50 ft. From Fig. 18 it is noted that initial no load void ratio (e_1) is 0.75, whereas the void ratio for the material after it has been consolidated under a load of 4 tons per sq ft is 0.67.

$$S = \frac{(0.75 - 0.67)50}{1.75}$$

$S = 2.28$ ft, which is theoretically the amount which should be added to the proposed height of the dam in order to compensate for settlement after the completion of the process of consolidation. In an actual case a number of such tests would be made at various elevations in the foundation.

The above method is approximate but considerably on the safe side since it assumes, first, that the unit pressure on the foundation, at the center of the dam, corresponds to the maximum height of the dam, whereas the center load spreads before reaching the foundation. Secondly, it assumes that the center load continues down through the foundation without spreading.

19. Optimum Moisture Content. If a dam is to be built by hydraulic fill methods, it matters little what the moisture content of the borrow pits may be, but if the dam is to be built by rolled fill methods, the moisture content may be of very great importance.

Borrow pits of sand and gravel if practically free from silt and clay may be extremely wet without causing trouble in rolled fill construction, but borrow pits for the impervious sections of rolled fill dams which contain silts and clays may cause serious trouble in a rolled fill if too wet. If the void ratio of the silt and clay in the borrow pit is higher than it would be in the earth dam after full consolidation and voids are filled with water, it may be impracticable to compact the material thoroughly by rolled fill methods.

If the material is too much on the wet side of the plastic limit (see Art. 2) it will certainly give trouble in securing proper compaction in the embankment.

In the construction of any rolled fill earth dam it is necessary to know what limitations to place on moisture of the borrow pit material being delivered on the fill. If the material is too dry, water may usually be added to bring it up to the desired moisture content, but if too wet serious trouble may be caused.

Tests and control methods developed by Proctor [7] have proved very useful in connection with rolled dam embankment containing silts and clays. Utilizing the given borrow pit material, specimen samples are made up in a compaction cylinder utilizing various percentages of moisture and using the standard method of compaction.

In the laboratory test a standard degree of compaction is obtained by using the 6 by 4 in. cylinder shown in Fig. 19. With the cap piece on, the cylinder is filled about half full of the material and then a 5½-lb tamping rod having a base area 2 in. in diameter (using the guide tube) is dropped on the material 25 times

[7] See R. R. PROCTOR, "Fundamental Principles of Soil Compaction," *Eng. News-Record*, Aug. 31, 1933, p. 245; "Description of Field and Laboratory Methods," *Eng. News-Record* Sept. 7, 1933.

for a height of 18 in., so that after three layers (approximately equal in height) have been placed and tamped the material will extend above the top of the cylinder proper and up into the cylinder cap about 1 in. The cap is then removed and the surplus is cut off. Density and moisture content are determined for the sample thus obtained.

The same material is utilized several times over in the test after additional moisture is added to the sample and thoroughly mixed through it. In some

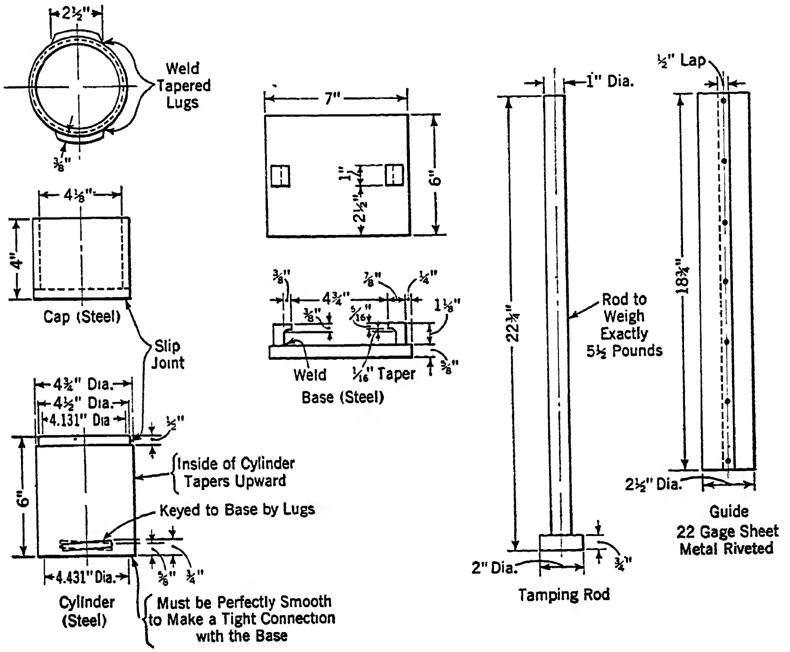

FIG. 19. Proctor compaction test equipment for determining optimum moisture content. (*From "Notes on Principles and Application of Soil Mechanics," U. S. Engineer Office, Fort Peck, Mont.*)

laboratories a mechanical arrangement is utilized for raising and dropping the tamper.

Fig. 20 shows a Proctor analysis for a clayey silt obtained with 25 blows. The optimum moisture content is the moisture content which gives maximum density (dry weight per cubic foot) after utilizing the standard method of compaction. In Fig. 20 it is shown that for this particular material the optimum moisture content is about 18 per cent. For a dry weight it is 107.6 lb per cu ft or for a wet weight 127.2 lb per cu ft.

It will be noted from Fig. 20 that for all densities except the one which gives optimum moisture content there are two moisture contents which give the same density. Thus, take a wet density (weight per cubic foot of embankment as placed) of, say, 124 lb per cu ft. From Fig. 20 this may be obtained either with a

moisture content of 16.8 per cent or 20.4 per cent. By using more than 25 blows a greater density can be obtained and the optimum moisture content will be less.

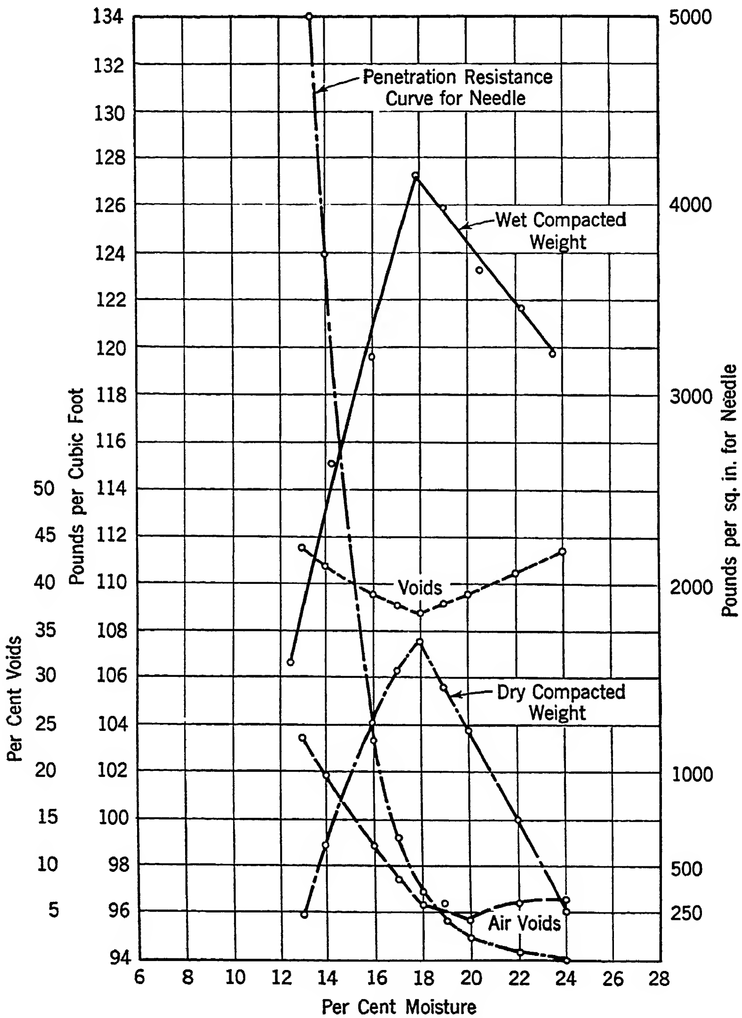

FIG. 20. Proctor analysis for a clayey silt.

In the field the indication of the laboratory test is followed as closely as may be. The author believes that in general it is better to keep a bit on the dry side of the laboratory indications. Thus in the above case if 124 lb per cu ft was the wet unit weight of the embankment, he would aim for a moisture content of 16.8 per cent rather than 20.4 per cent. (See Arts. 12, 19, and 30 of Chapter 19.)

In connection with compaction of rolled fills of plastic materials, it is not

always desirable to obtain the maximum possible density. It is better not to have density of such fills greatly exceed the density which it would reach at 100 per cent consolidation for a fill of that height and material.

Proctor also developed a soil plasticity needle in the head of which a spring is utilized to indicate the pressure required to force a needle of given base area [8] into the soil at a velocity of ½ ft per sec. By calibrating the needle against the compaction test it is possible to use the needle reading as an index to the moisture content for any given material. From moisture content, unit weight may be determined for such curves as those of Fig. 20.

In Fig. 20 is shown the calibration of a Proctor needle for the particular material utilized in that test. The Proctor needle is a very useful tool for inspectors on some embankments. The presence of gravel or small stones in the embankment makes the reading on the Proctor needle less reliable, and other quick methods for determining density and moisture content in the field have accordingly been devised. (See Art. 19, Chapter 19.)

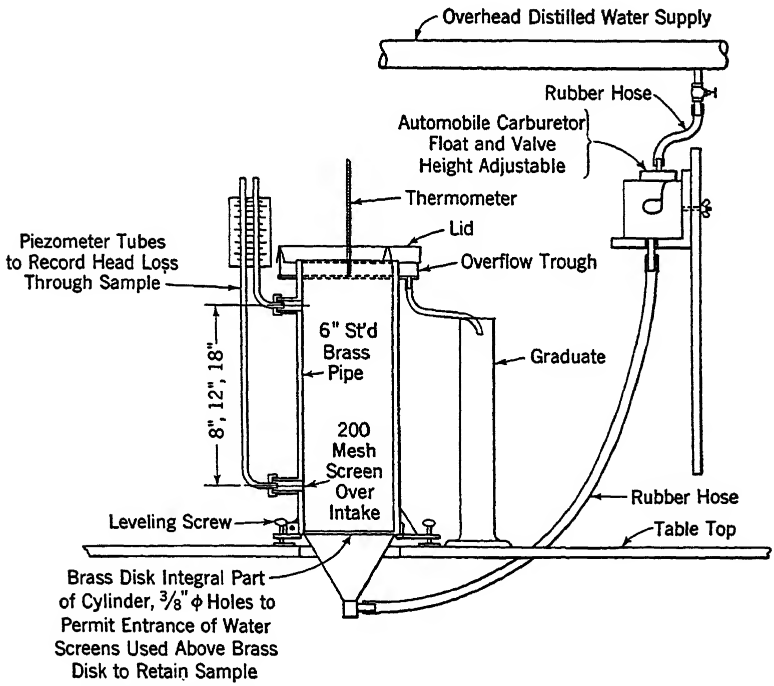

FIG. 21. Typical constant head permeameter. (*From "Notes on Principles and Application of Soil Mechanics," U. S. Engineer Office, Fort Peck, Mont.*)

20. Permeability. All soils are permeable because it is possible for water to pass through the pores or interstices of the soil mass. Thus silt is more permeable than clay, and sand is more permeable than silt.

[8] Usual areas for the needle are $\frac{1}{20}$th, $\frac{1}{10}$th, and $\frac{1}{5}$th sq in

The determination of the permeability of any given soil is relatively simple in principle, but a carefully worked out technique is just as essential here as with other soil tests. For instance, in order to obtain accurate and consistent results, it is desirable to avoid entraining any air in material being tested and it is also desirable to utilize only water from which the entrained air has been removed.

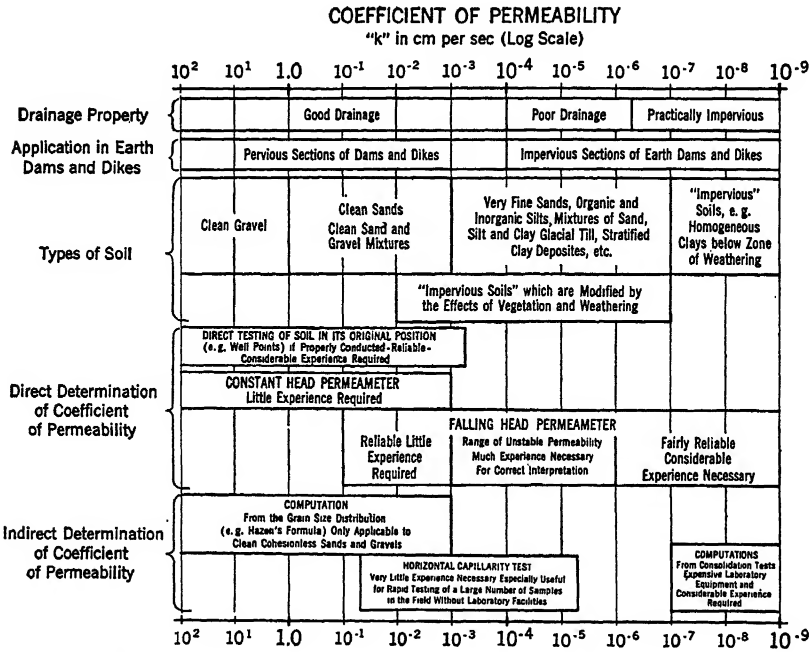

FIG. 22. Range in coefficient of permeability and application of methods for determining it. (*From "Notes on Soil Testing for Engineering Purposes," by A. Casagrande and R. E. Fadum, Graduate School of Engineering, Harvard University.*)

Both constant head and variable head permeameters are used and have their particular application. The constant head permeameter is suitable for use with gravels, sands, and coarse silts. These are the materials whose permeability we are most interested in, and fortunately tests with the constant head permeameter are relatively simple. Fig. 21 shows a typical constant head permeameter.

21. Flow of Water Through Soils. The flow of water which takes place through soils is extremely slow as compared with flow through pipes and channels. Flow through soils is usually referred to as laminar flow, in which velocity is directly proportional to the hydraulic gradient instead of the square of the velocity being proportional to the gradient as in turbulent flow.

Many experimenters have proposed formulas for expressing the relationship between flow through soils and the slope of the hydraulic gradient.

The Darcy formula [9] is generally used for expressing the flow relationship through soils.

$$Q = kiA \tag{5}$$

where Q = discharge in given unit of time,
i = hydraulic gradient,
A = area of soil mass through which flow takes place,
k = "coefficient of permeability" for the given material,

$$i = \frac{h}{l} = \frac{\text{difference in head}}{\text{length of path}}$$

k is the discharge through the unit area at unity hydraulic gradient. All factors affecting flow inherent in the nature of the material are comprised in the constant k.

Velocity through soil is

$$v = ki \tag{6}$$

where v is gross velocity over cross-section, including both soil particles and voids. If velocity of water through interstices is desired,

$$v_1 = \frac{ki}{P} \tag{7}$$

where v_1 = net velocity of water through interstices. Porosity P is used as a decimal and k and i have meanings given above.

22. Coefficient of Permeability. Several factors affect the value of k, the coefficient of permeability used in Eqs. 5, 6, and 7 for any given material. Among these factors are:

(*a*) The size and grading of particles.
(*b*) The density of the material as measured by porosity (or void ratio).
(*c*) The temperature of the water.
(*d*) The presence of organic matter.
(*e*) The presence of colloidal material.

The value of the coefficient of permeability should be determined by test for the materials at the dam site and in the borrow pits. The value of k is of greatest importance for gravels, sands, and silts. For the clays it is so small anyway that its exact value is not usually a matter of great importance.

In Fig. 23 are shown several mechanical analyses of a loess at Kingsley Dam. It will be noted that they are all much alike in grading. Fig. 24 shows the influence of density on the permeability of the same samples of loess.

Fig. 22 shows graphically the range in the coefficient of permeability for different materials as well as the methods applicable for determining the coefficient for the various materials.

With many alluvial deposits the permeability coefficient in a horizontal direction may be several times that in a vertical direction, but some loess deposits

[9] Derived by H. Darcy at Paris in 1856 to express results of his experiments.

on the other hand are many times as pervious in a vertical direction as in a horizontal direction. The coefficient of permeability should be determined experimentally for the various materials at the site of an earth dam and for the borrow pit material which may be used in constructing the dam. Nevertheless, it is frequently necessary to make estimates of seepage or ground-water flow where percolation tests are not available. In such cases it is assumed that mechanical

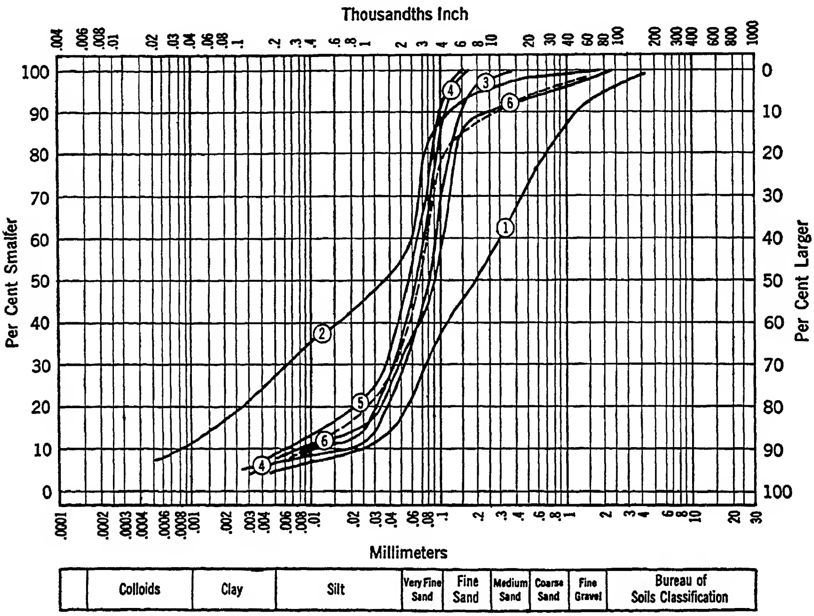

FIG. 23. Mechanical analyses of sandy loess used in core of Kingsley Dam, Ogallala, Nebr. (*Courtesy W. J. Turnbull.*)

analyses are obtainable because they can be made so quickly and simply. Table 2 gives the approximate permeability coefficient k for various soils from clay to fine sand.

The table represents the approximate average conditions met in the field for water-deposited materials and is based on several hundred percolation tests at Zanesville, Fort Peck, Kingsley, and Quabbin Dams.

As already indicated, no degree of accuracy can be expected unless the permeability coefficient is determined by carefully controlled experiments.

While size of particles and grading are the primary factors determining the value of the coefficient of permeability, density and temperature have a marked effect. The higher the temperature the greater the flow, and the denser the material the less the flow, other things being equal. These factors were not considered in the construction of the above Table 2.

TABLE 2

APPROXIMATE PERMEABILITY COEFFICIENTS OF VARIOUS SOILS
BASED ON 20 PER CENT SIZE

(1) 20% Size (mm)	(2) Coefficient of permeability $k \times 10^{+4}$ (cm per sec)	(3) Coefficient of permeability $k \times 10^{+4}$ (ft per min)	(4) Coefficient of permeability k (ft per min)	(5) Coefficient of permeability k (ft per yr)	(6) U. S. Bureau of Soil Classification
0.005	0.030	0.059	0.0000059	3.10	Coarse clay.
0.01	0.105	0.206	0.0000206	10.84	Fine silt.
0.02	0.400	0.787	0.0000787	41.40	Coarse silt.
0.03	0.850	1.675	0.0001675	88.20	
0.04	1.750	3.450	0.0003450	181.50	
0.05	2.800	5.510	0.0005510	290.00	
0.06	4.60	9.060	0.0009060	477.0	Very fine sand.
0.07	6.50	12.80	0.001280	673.0	
0.08	9.00	17.75	0.001775	935.0	
0.09	14.00	27.60	0.002760	1,450.00	
0.10	17.50	34.50	0.003450	1,815.0	
0.12	26.00	51.30	0.005130	2,698.0	Fine sand.
0.14	38.00	75.00	0.007500	3,940.	
0.16	51.00	100.00	0.010000	5,256.	
0.18	68.50	135.00	0.013500	7,100.	
0.20	89.00	175.00	0.017500	9,200.	
0.25	140.00	276.00	0.027600	14,500.	
0.30	220.00	434.00	0.04340	15,780.	Medium sand.
0.35	320.0	630.00	0.0630	33,150.	
0.40	450.0	886.00	0.0886	46,600.	
0.45	580.0	1,142.0	0.1142	60,000.	
0.50	750.0	1,480.	0.1480	77,800.	
0.60	1,100.	2,160.	0.2160	113,500.	Coarse sand.
0.70	1,600.	3,160.	0.3160	166,200.	
0.80	2,150.	4,240.	0.4240	223,200.	
0.90	2,800.	5,520.	0.5520	290,300.	
1.00	3,600.	7,100.	0.7100	373,500.	
2.00	18,000.	35,400.	3.540	1,860,000.	Fine gravel.

NOTE: This table represents a very rough approximation of average conditions in the field. A difference in density, temperature, or porosity may account for a wide difference in the coefficient of permeability. The 20 per cent size is that size than which 20 per cent of the sample is smaller and 80 per cent coarser.

Slichter [10] expresses in a very complete manner the factors causing variation in the coefficient of permeability.

> It is seen . . . that the quantity of water transmitted by a column of soil not only depends upon the length of the column and the head of water as expressed by Darcy's law, but varies in a most remarkable way with the effective size of the soil grain, with the temperature of the water, and with the porosity. Since the flow varies as the square of the size of the soil grain this element in the formula has a most important effect, as doubling the size of the soil grain will quadruple the flow of water. Thus, the flow through a sand whose effective size of grain is 1 mm is 10,000 times the flow through a soil whose effective size of grain is 0.01 mm. The variation of flow with temperature is also important, as the flow at 70° F is about double that at 32° F. The variation in porosity is quite as important as the variation in temperature. . . . If the two samples of the same sand are packed so that their porosities are 30 per cent and 40 per cent, the flow through the latter sample will be about 2.6 times the flow through the former sample.

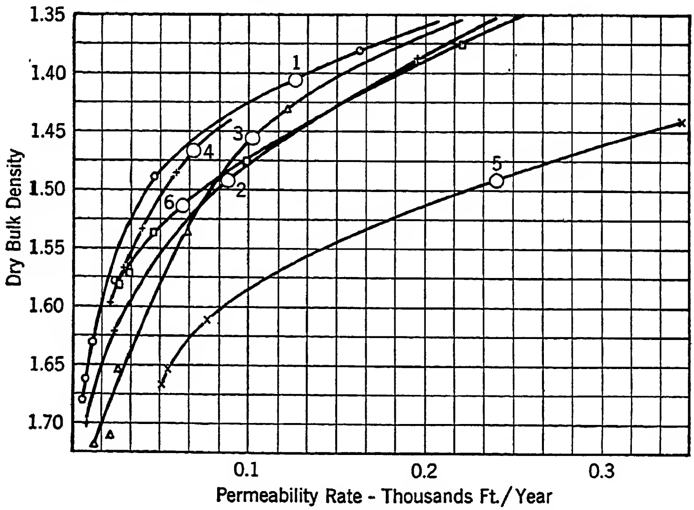

FIG. 24. Curves showing variation in permeability with density. Sandy loess, Kingsley Dam, North Platte River, Nebr. Numbers on curves indicate same material as represented by corresponding mechanical analyses in Fig. 23. "Dry bulk density" is ratio of the weight of unit volume of material (dry) to the weight of unit volume of water. Thus if "dry bulk density" is 1.5, dry weight of material per cu ft is 62.5 × 1.5 = 93.75 lb per cu ft. (*Courtesy W. J. Turnbull.*)

In Fig. 24 is shown the very considerable effect of density on the coefficient of permeability k. From the figure and the results of many tests, including those cited by Slichter, it is evident that difference in density alone may sometimes account for a wide difference in the value of the permeability coefficient.

23. Determination of Permeability Coefficient by Thiem Method.[11] This method, properly executed, permits the determination of the coefficient of per-

[10] C. S. SLICHTER, "The Motions of Underground Waters," *U. S. Geol. Survey Water-Supply Paper* 67, p. 25.

[11] G. THIEM, *Hydrologische*, Leipzig, 1906.

meability of the underground in the field and thus possesses certain advantages over laboratory methods where the permeability is determined on a small sample and the results often later applied over an extensive area.

The method [12] consists in sinking a well casing into or through the pervious stratum for which it is desired to determine the coefficient of permeability, inserting therein a well pump and operating the pump until the underground water surface becomes practically constant and then taking the measurements required by the formula.

The observation wells generally consist of small-diameter (say 1¼ in.) perforated pipe or well points. In general, observation wells should be far enough

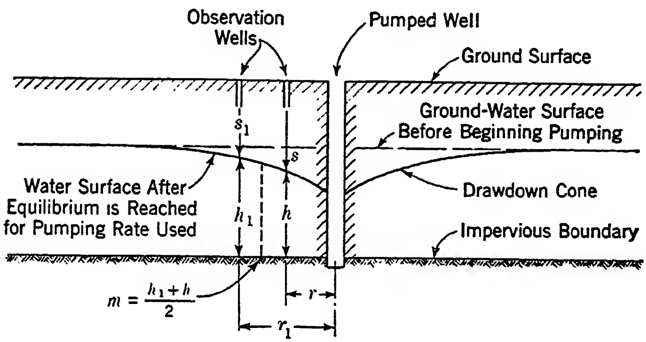

FIG. 25. Factors in Thiem formula for determining coefficient of permeability from field pumping tests.

from the pump well to avoid the steeper portion of the drawdown curves, which are the hydraulic gradients of the various filaments of flow. Usually a considerable number of these observation wells (10 to 50) are set on various radii from the pumped well because with a number of such wells a better picture of the average permeability of the underground may be obtained.[13]

The pump well casing should seldom be less than 12 in. in diameter, and larger sizes up to 24 in. may be advantageous. When pumping is started the water table draws down around the pump casing in the form of an inverted cone (see Fig. 25). Eventually, unless the pumping capacity is excessive in relation to the permeability of the underground, with steady pumping this cone becomes stable and the flow to the casing is then at the same rate as the pumping. It is at this point that the measurements should be taken. It is obvious that continuous operation of the pump is extremely desirable for accuracy. Therefore a duplicate installation of pumping equipment is desirable, so that if something goes wrong with one pumping outfit the other can be started. It is not desirable to depend on suction pumps because the drawdown may possibly be enough to

[12] See LELAND K. WENZEL, "The Thiem Method for Determining Permeability of Water-Bearing Materials," *U. S. Geol. Survey Water-Supply Paper* 679-A.

[13] See STANLEY M. DORE, "Permeability Determinations, Quabbin Dams," *Trans. Am. Soc. Civil Engrs.*, 1937, p. 682.

cause trouble with the suction before the cone reaches a condition of stability. Well pumps or centrifugal pumps which fit inside of the pump casing are suitable.

The Thiem formula modified to utilize units consistent with those used in this book is

$$k = \frac{q \log_{10} \frac{(r_1)}{r}}{20.4m(s - s_1)} \tag{8}$$

where k = the permeability coefficient in feet per minute (strictly cubic feet per minute per square foot on a 1 to 1 gradient) at a temperature of 54° F,

q = the rate of pumping in gallons per minute. r_1 and r are the distances of two observation wells from the pumped well in feet,

m = the average vertical thickness of the saturated portion of the water-bearing bed between the two observation wells and as indicated in Fig. 25 is equal to $\frac{h_1 + h}{2}$,

s and s_1 = drawdowns at the two observation wells, in feet.

Fig. 25 indicates the various quantities and measurements.

With a single installation the value of k may be determined between 20 or more pairs of observation wells.

The Thiem method applied by engineers experienced in making such observations is one of the most accurate methods of determining the actual permeability of the underground. It has been used successfully at several of the projects with which the authors have been connected, including Kingsley Dam on North Platte River, Nebraska, and Franklin Falls Dam, Pemigewasset River, New Hampshire.

Once the coefficient of permeability is determined, the seepage under the present or future hydraulic gradient may be computed with a fair degree of assurance.

24. Example in Use of Thiem Formula. Referring to Fig. 25, it will be assumed that after the cone has reached a condition of stability, measurements of the various factors have been taken and have been found to have the following values:

$$\begin{aligned}
q &= 1328 \text{ gal per min},\\
r &= 100 \text{ ft},\\
r_1 &= 150 \text{ ft},\\
r_1/r &= 1.5,\\
\log 1.5 &= 0.176,\\
m &= 45.82 \text{ ft},\\
s &= 4.73 \text{ ft},\\
s_1 &= 4.01 \text{ ft},\\
s - s_1 &= 0.72 \text{ ft}.
\end{aligned}$$

Substituting in the Thiem formula, we have

$$k = \frac{1328 \times 0.176}{20.4 \times 45.8 \times 0.72}$$

$$k = 0.347 \text{ ft per min}$$

From Table 2 it is evident that the material in the underground is a coarse sand, very likely a sand gravel.

25. Electrolytic Determination of Permeability Coefficient. A method of determining the permeability coefficient of the underground through the determination of the differences in electrical resistivity between ground water in its natural condition and the same water when charged with a heavy salt solution was devised by Slichter.[14] The method has the advantage that it is usually very much less expensive than the pumped well Thiem method, the cost frequently being quite insignificant. It has the disadvantage that any single test covers a relatively small area. W. J. Turnbull describes the use of this method in determining the permeability of the foundation at Kingsley Dam, on North Platte River, Nebraska, as follows:

> For making the field tests, a master well consisting of an 8-ft section of 4-in. diameter, perforated galvanized pipe was put down into the sand-gravel at Station 26± on the dam center line. Twelve observation wells were placed on the downstream (east) side of the master well; one well being placed at a distance of 2.5 ft; six wells being placed along a circle of 5 ft radius, with the master well as the center, and the remaining five wells being spaced regularly along a circle of 10 ft radius.
>
> The bottom 4 ft of the master well was perforated so as to allow free movement of water into and out of it. Each of the observation wells consisted of a 1¼ in. sandpoint and a section of galvanized iron pipe driven to a depth of 8 ft. The field setup of instruments was similar to that employed by Mr. Slichter. This setup consisted of a 6 volt storage battery and a direct current ammeter connected in series with the master well casing and the casing of any one observation well. The ground then formed the closing link in the series circuit. Then as the movement of the ground water carried the electrolyte from the master well towards the observation wells, the resistance of the ground circuit decreased resulting in an increase in the ammeter current reading. The wire connecting the ammeter to the observation well casing was fixed so that it could be easily connected to any desired observation well. In addition to taking readings between the master well casing and the observation wells, the battery and ammeter were arranged so that they could be connected in series with the casing of any observation well and a center electrode placed in that well. This center electrode consisted of a 3-ft brass rod with four wooden spools spaced evenly along its length to insulate it from the well casing. This electrode was also fixed so that it could be easily moved to any desired observation well. The reason for taking current readings within the observation well was to check the first procedure mentioned. In this latter procedure the time when the salt reached the well was indicated by a sudden increase in the ammeter reading. In the first procedure, the increase of the ammeter reading was slower, reaching a maximum when the maximum concentration of the salt

[14] C. S. Slichter, "Field Measurements of the Rate of Movement of Underground Waters," *U. S. Geol. Survey Water-Supply Paper* 140.

reached the downstream well. For charging the master well, a cloth sack approximately 5 ft long and 3 in. in diameter and filled with dry salt was employed. The sack held approximately 10 lb of dry salt. This salt charge was a 50-50 mixture of common salt (NaCl) and ammonium chloride (NH_4Cl). After filling the sack, it was tied at approximately every 6-in. point to prevent the salt from settling to the bottom of the sack as the salt dissolved thus keeping a uniform concentration moving out from the well for the full length of the perforations. New sacks of salt were introduced into the well as the previous one became empty. The test was concluded after 40 hr.

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CHAPTER 17

EARTH DAMS—GENERAL PRINCIPLES OF DESIGN

1. General. The general principles which should govern the design and construction of earth dams have been well understood by those engineers who have given special attention to the subject for many generations, but until quite recently their design has been very much more of an art than a science. The development of the science of soil mechanics has placed a valuable tool in the hands of the engineer which when properly used enables him to design and build earth dams with greater assurance and economy than was previously the case.

Unfortunately an earth dam appears to the uninitiated as a much simpler structure than it really is. Avoidable failures have riveted attention on the earth dam as an engineering structure, and today thorough investigations, studies, and tests are made of the foundation and of the materials available for construction, as discussed in detail in Chapters 1 and 16.

2. Utilization of Soil Mechanics. In recent years much of the advance in the art of designing and building earth dams has been due to the development of the science of soil mechanics. Dam engineers are rapidly becoming more scientific and are less rigidly bound by precedent provided that the proposed departure from precedent can be proved scientifically correct and also economically practicable. The change is all in the right direction so long as there is a proper balance between the practical and the theoretical.

Precise and highly mathematical methods for the design or analysis of the various features of an earth dam are seldom justified because we know in advance that the assumptions which must be made are necessarily sufficiently wide so that no precision is justifiable. Consequently the engineer whose function it is to investigate, design, and construct will, in most cases, do well to confine himself to the simple, less precise methods of design and analysis.

"Soil mechanics" is really a new name for an old science. Rankine, Coulomb, and Darcy were certainly soil mechanicians of several generations ago, but they probably would not have recognized the title. The fundamental principles of soil mechanics, like those of hydraulics, are relatively simple. For most of the problems of design, suitable methods of analysis are available which are no more complicated than the more usual methods of design of, say, reinforced concrete structures.

What has happened in recent years is that, very largely due to the magnetism, persistence, and ability of Dr. Karl Terzaghi and some of the men whom he has trained, there has been a renewed interest in the investigation and study of the physical properties of soils and their application to engineering structures. This

has resulted in providing new and better methods for determining the stresses and factors of safety in earth foundations, dams, and embankments.

3. Foundation of Earth Dams. It is possible to construct a safe earth dam on almost any foundation which is available provided that the foundation has been thoroughly explored and tested in the manner discussed in Chapters 1 and 16 and the design adapted to the conditions thus revealed.

Ledge rock foundations for small dams seldom give cause for concern except that in some cases they may require grouting. Foundations of earth dams are often more or less recent alluvial deposits which have not been consolidated under any material load. Coarse sands and gravels in the foundation of an earth dam give no trouble with regard to stability because, although they may not be consolidated, they will promptly consolidate as the load is applied (see Art. 12 to 17, Chapter 16).

For very fine and uniform sands great caution is necessary. If less dense than "critical density" they may, when saturated under load, flow almost like a liquid if activated by some disturbance as, for instance, an earthquake or even blasting or the passage of trains. As shown in Art. 13, Chapter 16, it is possible to consolidate such a foundation to a point where it is more dense than the critical density and is thus no longer subject to flow on disturbance.

A plastic clay foundation in an earth dam is usually the type of foundation which requires the greatest amount of study and investigation in order to obtain unquestionable safety (see Arts. 8 and 14, Chapter 16). Frequently extremely flat slopes must be used for the earth dam built on such a foundation in order to keep the stresses in the foundation sufficiently less than the strength of the material to provide a suitable factor of safety.

The condition of the foundation is one of the important factors in choosing a dam site. Other things being equal, we would choose the dam site with the best foundation conditions. Over-all economic considerations should govern the choice of site. For instance, one might choose a site having a foundation of plastic clay to a considerable depth because in spite of the flat slopes that would be required the site would permit a much shorter dam, so that the total cost of the dam at this site would be materially less than at another location with a smaller cross-section.

4. Materials of Construction. Earth dams have been built successfully of loose rock, of gravel, of sand of all degrees of fineness, silt, rock flour, and clay. The materials for a concrete dam, cement and aggregate, may come and frequently do come from a source located a great distance from the site. Because of the very much greater quantities of materials involved for an earth dam, the materials must come from borrow pits and quarries close to the site to avoid prohibitive cost.

It is the engineer's business to find out the character, properties, and quantities of the various materials in foundation and borrow pits. Once these are known and thoroughly understood, it is almost always possible to design and construct an entirely safe dam from the available materials. Of course, if the site were in a rock canyon with no earth at all nearby, very little time would be wasted in the consideration of an earth dam for such a site. One would proba-

bly build either a concrete or a rock-fill dam. While it is desirable to have available ample quantities of both pervious and impervious materials, it is entirely practicable to construct a dam almost entirely of either class of material.

5. Design to Suit the Available Materials.[1] In the interest of economy the design of an earth dam should be adapted to the utilization of the materials available at or near the site. Thus, if near the site there is nothing available but sand, then the adopted design should utilize this sand for the bulk of the dam, limiting the imported material of concrete, clay, or silt for providing an impervious member to the minimum required.

In Fig. 1 is shown a suitable design for a site where there is nothing available except sand gravel. The nearest impervious material is a sandy clay, which

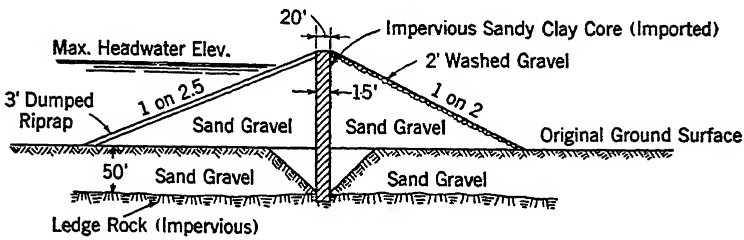


FIG. 1. Suitable design for a site where only sand gravel is available and foundation is pervious to depth of 50 ft.

Note. Concrete core wall and/or steel sheet piling might be substituted for the sandy clay core, but would not be as desirable.

is located 10 miles away. The difficulties of excavation and transportation are such that the cost of sandy clay in place is $5 per cu yd, and consequently the use of this material is to be kept at a minimum.

Accordingly, a trench with 1 on 1 side slopes is excavated to ledge rock a vertical distance of 50 ft. A 15-ft width of the ledge rock is cleaned off very carefully and a 15 ft wide section of the impervious sandy clay is bonded to it. The rest of the trench is then refilled (in the dry) and thoroughly compacted, using sheep's-foot roller and/or air tampers. For a width of 15 ft in the center of the trench only the impervious sandy clay is used, but outside of that limit, the refill is the sand gravel which forms the original foundation.

Above the base of the dam the 15 ft wide core is extended, being placed as a part of the 8-in. horizontal layers in which the dam is being carried up and compacted.

A thin reinforced concrete core wall 1 to 3 ft thick might be substituted for the sandy clay core if less expensive, but it would not be quite as desirable because the sandy clay core would adjust itself more readily to any slight movement and would also be tighter than the concrete.

The drainage conditions for such a design would be excellent. The cutoff through the foundation would make the downstream portion of the sand gravel

[1] See also JOEL D. JUSTIN, *Earth Dam Projects*, John Wiley & Sons, Inc., 1932, pp. 88 and 115.

forming that foundation available as a drain for the slight amount of seepage water which did get through the core. After passing through the core, the seepage line will drop promptly to the foundation. The upstream slope is somewhat flatter than the downstream slope in order to take care of possible drawdown pressures.

Fig. 2 shows a design suitable for a site where both clayey silt and coarse pervious sand are available in adequate quantities in borrow pits near the site. As in Fig. 1, both drainage and stability conditions are favorable. The impervious

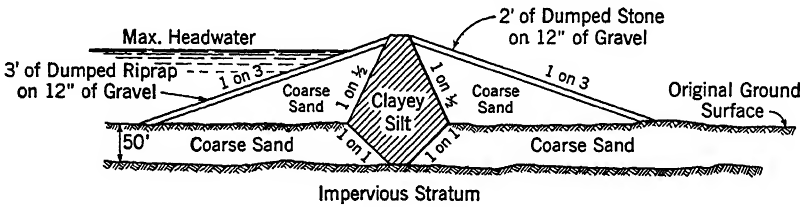


FIG. 2. Suitable design for a site where both clayey silt and coarse sand are available and where foundation is pervious for depth of 50 ft.

Note. Steel sheet piling might be substituted for foundation cutoff.

stratum 50 ft below the base of the dam has adequate shear strength. The foundation cutoff leaves the pervious foundation downstream from the cutoff available as a drain for such water as gets through the core or cutoff.

The design in Fig. 3 is suitable for a site where both clayey silt and coarse sand are available in adequate quantities and where the foundation is impervious (either ledge rock or consolidated clay). In this case it is evident that such water as does get through the relatively impervious central section of the dam must appear at the downstream face or toe because it cannot enter the founda-

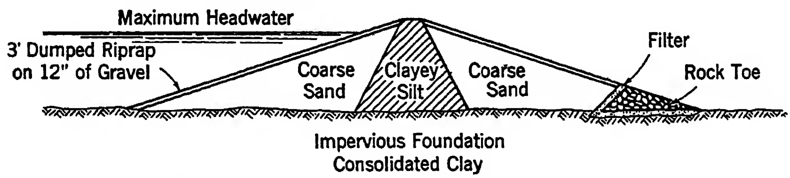


FIG. 3. Suitable design for a site where both clayey silt and coarse sand are available and where the foundation is impervious.

tion. Hence to prepare for this condition and avoid the possibility of sloughing due to saturation a rock toe with a filter consisting of smaller stones with gravel and sand just ahead of it to protect it against impregnation is utilized. Even without going into the design from a quantitative standpoint, it is evident that seepage will be entirely insignificant.

In Fig. 4 is shown a suitable design for a site where both sand gravel and clayey silt are available and where the foundation is highly pervious to a great depth. A blanket of clayey silt, which is very impervious as compared with the sand gravel of the foundation, is carried from the impervious core upstream under

the upstream shell and extended for a distance frequently 10 or more times the head upstream from the upstream toe of the dam. Such blankets cut down the seepage materially by forcing the water to pass through several times the distance which it would have to pass through without the blanket.

Also under conditions in Fig. 4 the foundation will be full of seeping water and provision to take care of seepage is provided by a filter layer at the base which is even more pervious than the sand gravel of the foundation.

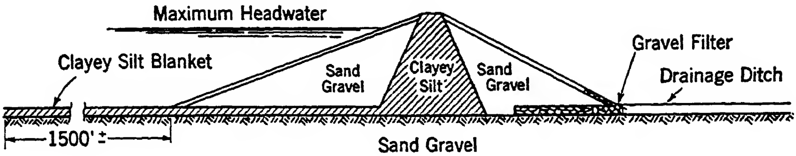


FIG. 4. Suitable design for a site where both sand gravel and clayey silt are available and foundation is highly pervious to a great depth.

In Fig. 5 is shown an earth dam design which is suitable for a site where the only material available is a silty clay and where the foundation consists of a silty clay which is highly unconsolidated.

In this case the upstream slope is flattened to take care of rapid drawdown, and also in many cases the flatness of both slopes is determined by the requirements for spreading the load so that the maximum unit stress induced in the foundation will be less than the shear strength of the plastic material in the foundation with a fair factor of safety.

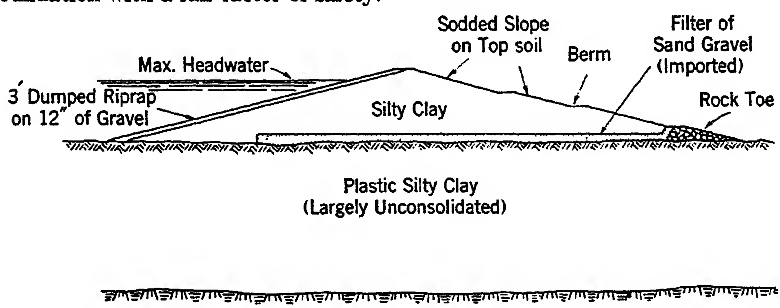


FIG. 5. Suitable design for a site where the only embankment material available is a silty clay and where the foundation consists of a silty clay which is largely unconsolidated.

It will be noted that a filter layer is placed on the foundation under the base of the dam except near the upstream toe. The filter layer of sand gravel may be "run of bank devoid of clay," as it will surely be vastly more pervious than the silty clay. This filter layer has two particular functions: (1) to provide drainage for the small amount of seepage and thus prevent any possibility of saturation of the downstream face and (2) by providing drainage for water squeezed out of the silty clay by the loading added during construction, it has a marked effect in accelerating consolidation and hence hastens an increase in shear strength of the foundation material.

TABLE 1

PARTIAL LIST OF EARTH DAM FAILURES

No.	Name	Location	Ht. (ft)	Date of Failure	Reference	Core	Remarks
1	Fort Peck *	Montana	225	1938	War Dept., Report on Slide, 1939	Silt hydr. fill	Foundation failure, low shear strength.
2	La Regadera *	Colombia, So. Amer.	108	1937	Personal records	Clay	Foundation failure in plastic clay.
3	Marshall Creek *	Kansas	70	1937	Report on Failure & Reconstruction 2/11/38, Chief of Engineers U.S.A.	Clay, silt	Foundation failure in plastic clay.
4	Clendening *	Ohio	62	1934	Personal records	Imperv. rolled material	Slight shear failure of plastic embankment.
5	Tappan *	Ohio	52	1934	U. S. Engineer Off., Zanesville, Ohio	Rolled material 25% clay	Movement from excess consolidation.
6	Belle Fourche *	S. Dakota	115	1933	*Eng. News-Record,* V. 111, p. 371	Rolled earth	Upstream slide, sudden drawdown.
7	Alexander	Hawaiian Islands	125	1930	*Eng. News-Record,* V. 104, p. 869	Hydraulic fill	Internal liquid pressure.
8	Balsam	New Hampshire	60	1929	*Eng. News-Record,* V. 102, p. 885	Concrete	Spillway discharge eroded toe caused sloughing.
9	Pleasant Valley	Utah	63	1928	*Eng. News-Record,* V. 101, p. 915	Puddle cutoff earth fill	Settlement and piping.
10	Table Rock Cove	Greenville, S. C.	140	1928	*Eng. News-Record,* V. 103, p. 934	Clay	Partial failure due to broken outlet pipe.
11	Puddingstone	California	50	1926	*Eng. News-Record,* V. 96, pp. 665–913	Conc. facing	Overtopping.
12	Apishapa	Colorado	115	1923	*Eng. News-Record,* V. 91, pp. 357–418	Concrete	Settlement cracks, caused piping.
13	Calaveras *	California	240	1918	Idem, V. 72, p. 692	Clay	Liquid pressure on shells.
14	Lower Otay	California	136	1916	Idem, V. 75, p. 334	Steel	Insufficient spillway.
15	Weisse, Passe River	Bohemia	42	1916	Idem, V. 77, p. 139	Steel	Seepage along conduit.

16	Lyman	Arizona	65	1915	Idem, V. 73, p. 764	Puddled	Piping and sloughing.
17	Horse Creek	Colorado	56	1914	Idem, V. 71, p. 828	None	Piping and sloughing.
18	Lake George	Colorado	..	1914	State Engr. Report	Puddled	Piping.
19	Hatchtown	Utah	65	1914	*Eng. News-Record,* V. 75, p. 60	....	Seepage along culvert.
20	Hebron	New Mexico	56	1914	Idem, V. 69, p. 629	....	Water through gopher holes.
21	Davis Reservoir	California	39	1914	Idem, V. 72, p. 106	....	Piping. No cutoffs on gate structure.
22	Sepulveda Canyon	California	65	1914	Idem, V. 74, p. 357	Rein. concr. core	Insufficient spillway.
23	Colorado Springs	Colorado	50	1912	Idem, V. 66, p. 223	....	Partial failure due to piping.
24	West Julesburg	Colorado	50	1910	State Engr. Report	None	Seepage along ledge rock.
25	Zuni	Black Rock, N. Mex.	70	1909	*Eng. News,* V. 62, p. 597	None	Hydraulic fill and rock piping.
26	Necaxa *	Mexico	193	1909	Idem, V. 60, p. 1	Clay	Sloughing during construction.
27	Debris Barrier No. 1, Yuba River	California	...	1907	Idem, V. 58, p. 609	....	Insufficient spillway.
28	Lake Avalon	New Mexico	48	1904	Idem, V. 54, p. 9	....	Piping.
29	Greenlick, Scottsdale Dam	Pennsylvania	60	1904	Idem, V. 52, p. 107	....	Piping.
30	Utica Reservoir	New York	70	1902	Idem, V. 48, pp. 225–290	....	Steep slopes, poor construction.
31	Lake Francis	California	50	1899	*Trans. A.S.C.E.,* V. 58, 1907, p. 196	None	Settlement and seepage along outlet conduit.
32	Snake Ravine	California	64	1898	*Eng. News,* V. 40, p. 242	Hydraulic	Poor construction.
33	Johnstown	Pennsylvania	72	1889	*Eng. Record,* V. 20, p. 30	None	Insufficient spillway.
34	Ashti *	India	58	1883	Wegmann, p. 234	Puddled	Partial foundation failure; low shear strength.
35	Swansea	South Wales, Great Britain	80	1879	*Sanitary Eng.,* V. 3, p. 437	Puddle	Piping.

* Slide or movement during construction, dam later repaired and completed.

In considering Figs. 1 to 5, it should not be assumed that these figures represent actual designs; they are merely types of designs suitable for the stated materials available and the given foundation conditions. Actual designs for the given condition should be arrived at on the basis of computation and analysis.

6. Failure of Earth Dams. The study of failures frequently contributes greatly to our knowledge in showing us how not to do things.[2] Overtopping due to insufficient spillway capacity is the most frequent cause of the failure of earth dams as shown by a study of more than 100 failures. Piping of one sort or another, according to the record, is responsible for a large number of failures (see Art. 35). A number of failures have been caused by insufficient shear strength in the foundation or in the embankment itself leading to slides of various sorts. Fortunately troubles of this sort generally occur during construction because if once successfully completed, the shear strength of embankment and foundation increases with age and further consolidation.

Particular attention is called to some of the more recent failures or construction accidents listed in Table 1 in connection with structures of considerable magnitude. (See also Arts. 12, 29, and 41 of Chapter 19.) Of five such recent failures, Fort Peck (item 1), Clendening (item 4), Tappan (item 5), Marshall Creek (item 3), and La Regedera (item 2), all except Clendening were foundation failures. All except Fort Peck (which was a hydraulic-fill dam) were constructed by rolled fill methods. The method of construction utilized, however, had nothing to do with the failure. The demonstrated deficiencies of all of these dams were corrected and today all of them are successful earth dams.

7. Requirements for the Safety of Earth Dams. The practical criteria for the design of earth dams may be stated briefly as follows: An earth dam should be so designed that

1. There is no danger of overtopping (i.e., sufficient spillway capacity and sufficient freeboard).
2. The seepage line is well within the downstream face.
3. The upstream face slope is safe against sudden drawdown.
4. The upstream and downstream slope is flat enough that, with the materials utilized in the embankment, they will be stable and show a satisfactory factor of safety by recognized methods of analysis.
5. The upstream and downstream slopes of the earth dam are flat enough that the shear stress induced in the foundation is enough less than the shear strength of the material in the foundation to insure a suitable factor of safety.
6. There is no opportunity for the free passage of water from the upstream to the downstream face.
7. Water which passes through and under the dam when it reaches the discharge surface has a pressure and velocity so small that it is incapable of moving the material of which the dam or its foundation is composed.
8. The upstream face is properly protected against wave action and the downstream face is protected against the action of rain.[3]

[2] In Chapter 1 of JOEL D. JUSTIN, *Earth Dam Projects*, John Wiley & Sons, Inc., New York, 1932, a number of earth dam failures are discussed and analyzed.

[3] Somewhat similar criteria for designs were included in *Earth Dam Projects*, John Wiley & Sons, Inc., New York, 1932, and in "The Design of Earth Dams," *Trans. Am. Soc Civil Engrs.*, Vol. 87, 1924, p. 1.

The principal purpose of stating the above criteria of design is to furnish a check list which the engineer can consult to help him make sure that he has considered all of the pertinent factors in the design of his dam. An earth dam designed to meet these criteria will prove permanently safe provided proper attention is given to the details of construction.

Methods for satisfying criteria 1, 2, 6, and 7 will be discussed in this chapter.

8. Safety Against Overtopping. An earth dam should be designed with the spillway capacity so great that there is no danger of overtopping. One frequent cause of the failure of earth dams is the use of spillways of insufficient capacity. A masonry dam with an insufficient spillway will generally stand overtopping to a considerable depth without serious damage, but with an earth dam, overtopping usually means failure. Many earth dams are in use that have spillways of insufficient capacity to care for floods, which are certain to come sooner or later.

A dam with a concrete core wall is as likely to fail from overtopping as one with a core of impervious soil, since once the downstream embankment is washed away, the concrete wall is not stable and will break or tip over.

After the greatest flood to be expected has been determined, the spillway should be designed to take such a flood with a fair factor of safety. Chapter 5, to which the reader is referred, deals with the subject of spillway requirements.

Engineers frequently speak of gross freeboard and net freeboard. Gross freeboard, more properly called "surcharge," is the vertical distance from the crest of the spillway to the top of the dam. Net freeboard is the vertical distance from reservoir surface to the top of the dam at the time that the spillway is discharging the greatest flood to be expected (according to the accepted hydrological studies). When freeboard is mentioned herein net freeboard is meant unless otherwise stated.

The net freeboard should be the sum of the heights of tides, seiches, wind setup, and the height to which waves will ride up on the upstream face plus a margin of safety in feet based on judgment. For a suitable method of computing these heights the reader is referred to Arts. 9 and 10, Chapter 7. The proper margin of safety to use is dependent on the degree of conservatism utilized in determining the greatest flood to be expected. If the available data on floods in the territory are extensive and if the methods employed to determine the greatest flood to be expected are extremely conservative, then this additional margin of safety may be omitted altogether. As indicating the conservatism of the recommended assumptions, in this connection it is pointed out that, for the larger drainage areas, the peak of the flood lags considerably behind the storm and this involves the coincidence of two contingencies which have a very slight chance of coincidence except on very small watersheds, i.e., maximum reservoir elevation and maximum wind velocity.

9. Seepage. Seepage takes place through and under all dams, both earth and concrete. The problem is to minimize and control seepage so that it will have no harmful effects. The character of the materials comprising the foundation and the embankment has a very important influence on seepage and its effects. Thus, just as a matter of contrast, the author knows of one dam where seepage is

at least 30 sec-ft, but it does no harm at all. On the other hand, the author also knows of another dam, of which the total seepage through and under it is less than 1 sec-ft, but the downstream face is saturated and sloughing has taken place, and consequently remedial measures are now under way.

For any dam of homogeneous material founded on an impervious base, the seepage will pass through the dam and will appear on the downstream face because there is no other place for it to go. This will happen regardless of the tightness of the embankment material. If the dam of homogeneous material is founded on a pervious foundation, seepage may still be expected to appear on the downstream face unless a cutoff has been constructed through the pervious foundation, thus permitting the downstream portion of pervious foundation to act as a drain. Of course, it is a simple matter to provide drainage so that the seepage does not reach the downstream face but will be taken care of and conducted to the downstream toe. In that case the dam would no longer be, strictly speaking, composed of homogeneous material.

10. Position of the Seepage Line. In an earth dam composed of material so coarse that capillarity has no influence, the "seepage line" is practically the line of saturation; i.e., the uppermost filament of flow. In this case and in all other cases, the seepage line may be defined as the *line above which there is no hydrostatic pressure and below which there is hydrostatic pressure.* This definition must be adhered to strictly whenever the words are used in connection with earth dams composed of any type of material. This is because, where the material is fine enough to be subject to a considerable depth of capillarity, there is saturation without hydrostatic pressure and also a usually negligible flow in the "capillary fringe" above the "seepage line."

It is because of this inadequacy in the literal definition of "seepage line" that the words "phreatic line" were coined. However, the two phrases can be considered synonymous if the definition of "seepage line" is considered as given above. The capillary fringe is of no particular significance in large earth dams except for special investigations, but it is of great interest in earth models of earth dams.

It is always desirable to be able to predict, at least approximately, the position of the line of seepage in the cross-section of a proposed earth dam. If this line is allowed to intersect the outside downstream face much above the toe more or less serious sloughing may take place and ultimate failure may result.

If we desire to draw a flow net for the dam the use of the seepage line as one of the boundaries greatly simplifies the procedure.

For an earth dam composed of homogeneous material located on a foundation of impervious material the seepage line will cut the downstream face above the base of the dam unless, of course, special drainage measures are adopted. The location of the seepage line in this case and the point at which it cuts the downstream face is dependent only on the cross-section of the dam. Its position is not influenced by the permeability of the material composing the dam so long as that material is homogeneous. The seepage line under the assumed conditions

has been shown to be fundamentally a parabola [4] with departures therefrom due to the local conditions of ingress and egress.

In Fig. 6

B_2 is a point on the seepage parabola extended to intersect the water surface.
A is the downstream toe of the dam. If the dam is composed of a relatively impervious core with a pervious shell, A is the downstream toe of the core.
C is the intersection of the seepage line with the downstream face of the dam (or core).
d is the horizontal distance from point B_2 to point A.
h is the vertical distance from point B_2 to point A and represents the head causing seepage.
a is the distance AC and represents the wetted portion of the downstream face.
α is the internal angle formed by the downstream discharge face and the horizontal base, as in Fig. 7a.
m is the horizontal projection of the wetted upstream slope.
k is the coefficient of permeability of the material comprising the dam (or core).

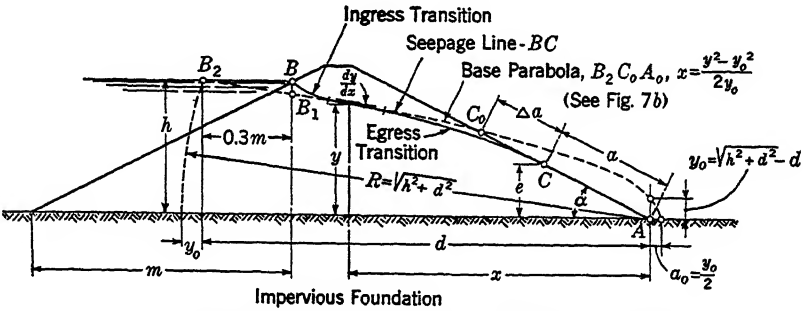


FIG. 6. Determination of seepage line. (*After A. Casagrande.*)

Calculations of seepage through soils are based on Darcy's law, given as Eq. 5, Art. 21, Chapter 16.

Casagrande [5] has shown that the computed seepage line for all but the smallest angles of discharge slope approximate quite closely the "base parabola" established by Professor Kozeny [5] for the case when α is 180°.

If in Darcy's Eq. 5 in Chapter 16 the area of cross-section, A, at any point along the base of the dam is represented by y and the hydraulic gradient, i, at that point is represented by the slope of the seepage line $\frac{dy}{dx}$ (see Fig. 6), then the seepage flow through the dam would be given by the equation

$$q = ky\frac{dy}{dx} \tag{1}$$

[4] See ARTHUR CASAGRANDE, "Seepage Through Dams," *J. New Eng. Water Works Assoc.*, June, 1937, p. 137.

[5] ARTHUR CASAGRANDE, "Seepage Through Dams," *J. New Eng. Water Works Assoc.*, June, 1937.

Kozeny proved that, for the case where $\alpha = 180°$, the seepage line could be represented by the equation

$$x = \frac{y^2 - y_0^2}{2y_0} \qquad [2]^{6}$$

which is a parabola with the focus at A intersecting a perpendicular at A at a distance y_0 from the base line (Fig. 6).

The parabola should be continued to intersect the water surface at point B_2 (Fig. 6), which has the coordinates $y = h$, $x = d$, where d equals the width of the base of the dam minus $0.7m$. If these values of x and y are substituted in Eq. 2, the value of y_0 becomes

$$y_0 = \sqrt{h^2 + d^2} - d \qquad [3]$$

The value of y_0 may be readily determined graphically [6] since it is the difference between the slant distance and the horizontal projection of the line AB_2, Fig. 6.

The point C_0 where the base parabola intersects the downstream face is easily found from the polar equation of a parabola which is

$$r = \frac{P}{1 - \cos\theta} \qquad [4]$$

where r = radial distance from focus to point on parabola,
P = intercept of parabola on normal to axis line through focus,
θ = angle of radial line with axis of parabola.

For the particular values y, r, and θ being considered

$r = a + \Delta a$ = slant distance from A to point of intersection of base parabola with downstream face,
$P = y_0$ = intercept of parabola on vertical line through focus,
$\theta = \alpha$ = angle of downstream face.

Thus it follows from Eq. 4 that

$$a + \Delta a = \frac{y_0}{1 - \cos\alpha} \qquad [5]$$

Reference to Fig. 6 shows that the intersection of the seepage line with the downstream face occurs at point C, a distance Δa below the point of intersection for the base parabola, C_0. Casagrande [7] has shown that the distance Δa varies with the slope angle α, becoming zero when $\alpha = 180°$. Fig. 7b gives the ratio of Δa to $a + \Delta a$ as determined by his graphical studies by means of flow nets. The lower end of the seepage line is completed by drawing in a transition curve from C to the base parabola by eye, as indicated in Fig. 6.

The upstream end of the seepage line is also sketched in by eye, a short transition curve being drawn to connect point B with the base parabola as shown in

[6] ARTHUR CASAGRANDE, "Seepage Through Dams," *J. New Eng. Water Works Assoc.*, June, 1937.

[7] Idem.

Fig. 6. This transition curve should start normal to the upstream face at the point of intersection with the free water surface. Where the upstream face of the dam or core has a very steep slope, the transition may be a reverse curve.

There are several equations which may be used to calculate the seepage flow. One of the simplest of these is derived for the case where the seepage line is represented at its lower end by the base parabola ($\alpha = 180°$).

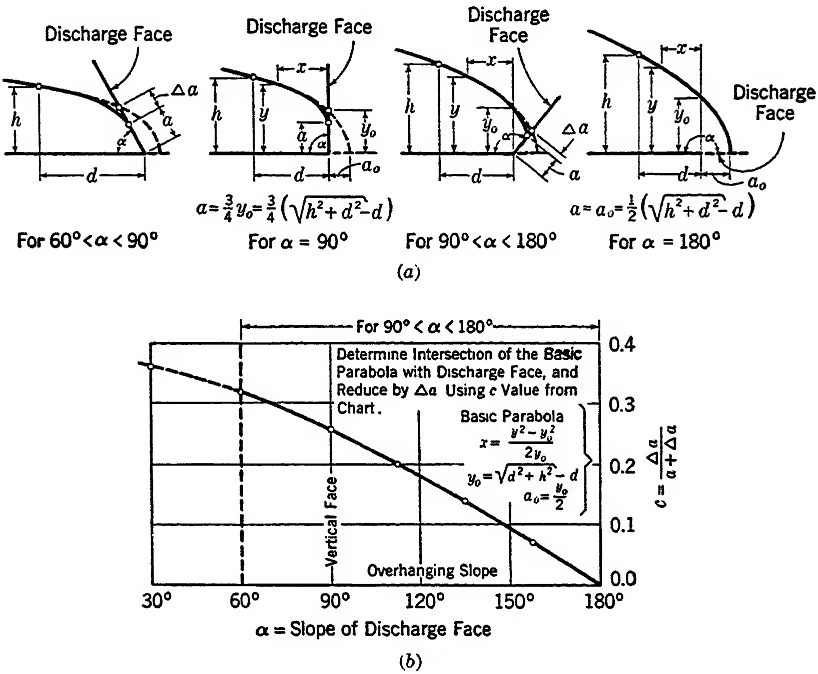


FIG. 7. Diagrams for determining Δa and a in Fig. 6 when $a + \Delta a$ and slope of discharge face are known. (*From "Seepage Through Dams," by A. Casagrande, p. 149, J. New Engl. Water Works Assoc., June 1937.*)

From Eq. 2 of the base parabola

$$y = \sqrt{2xy_0 + y_0^2} \qquad [6]$$

and

$$\frac{dy}{dx} = \frac{y_0}{\sqrt{xy_0 + y_0^2}}$$

If the values of y and dy/dx are substituted in the Darcy equation as given in Eq. 1

$$q = k\sqrt{2xy_0 + y_0^2}\,\frac{(y_0)}{\sqrt{2xy_0 + y_0^2}}$$

$$q = ky_0 \qquad [7]$$

Eq. 7 gives the seepage flow for the case where $\alpha = 180°$. Inspection of Fig. 7*a* shows that the mean length of path and the cross-section area of the seepage flow is but slightly different for angles less than 180° but greater than 30°. If the value of y_0 given in Eq. 3 is substituted in Eq. 7

$$q = k(\sqrt{h^2 + d^2} - d) \qquad [8]$$

For most cases encountered in earth dam design, the seepage may be calculated with sufficient precision by Eq. 8. Where the angle of the discharge face, α, is less than 30°, the following equation may be used:

$$q = ka \sin^2 \alpha \qquad [9]$$

when

$$a = \sqrt{h^2 + d^2} - \sqrt{d^2 - h^2 \cot^2 \alpha} \qquad [10]$$

Eq. 9 gives somewhat smaller values for q than Eq. 8 in the range for which it is suitable.

In Art. 16 another approximate equation is used to calculate the seepage flow and an example is worked out to show the closeness of agreement with Eq. 8.

11. Seepage Line Where Vertical and Horizontal Permeability Differ. Soils deposited by water and soils placed in earth dams, in rolled dams, or in hydraulic fill dams may show a wide difference between their vertical and horizontal permeability. (See also Art. 19.) For water-deposited soils in nature, horizontal permeability may be 4 to 20 times the vertical permeability. In such cases where one desires to locate the seepage line and/or draw the flow net a transformed section may be utilized. To make the transformation, multiply the actual

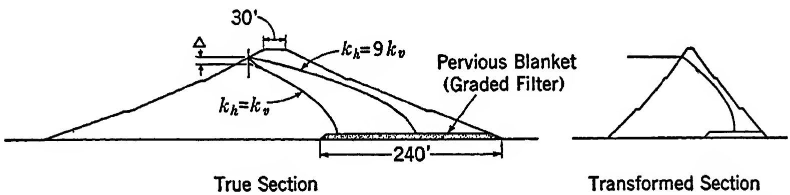


FIG. 8. Transformation method for analysis of stratified sections. (*From "Seepage Through Dams," by A. Casagrande, p. 158, J. New Engl. Water Works Assoc., June 1937.*)

horizontal dimensions by $\sqrt{k_v/k_h}$ where k_v equals permeability coefficient of the material in a vertical direction and k_h equals permeability coefficient of the same material in a horizontal direction.[8] The line of seepage and flow net may thus be determined in the same manner as for a soil which is homogeneous and isotropic, and the dimensions including the seepage line and/or the flow net may be transposed back to the true section. Such a transformation for a case where the horizontal permeability is 9 times the vertical is shown in Fig. 8.

12. The Flow Net. (See also Art. 14, Chapter 3.) The flow net has many applications, but in the present discussion its application will be confined to the two-dimensional flow of water through soils. The flow net consists of two sets of curvilinear lines at right angles to each other. The intersections form homol-

[8] Idem, p. 151.

ogous rectangles, and it is convenient in many problems to so choose the factors that the rectangles are squares.

The flow lines comprising one set are the paths which would actually be taken by filaments of water flowing through the soil. The other lines which are perpendicular to the flow lines are known as the equipotential lines because the elevation of water surface within any piezometer, placed at any point on the same equipotential line, will be the same. Any point on one of these lines will show the same hydraulic pressure (or head) loss as every other point on this line. The distance between any two adjacent equipotential lines represents the same loss of head as the distance between any other two adjacent equipotential lines.

Similarly the distance between any two adjacent flow lines will represent the same increment of flow as the distance between any other two adjacent flow lines.

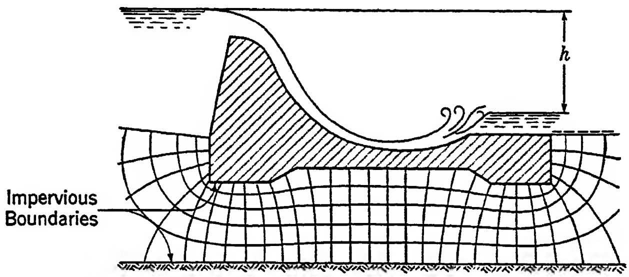


Fig. 9. Flow net under a concrete dam on sand. *(From "Uplift and Seepage Under Dams on Sand," by L. F. Harza, Trans. Am. Soc. Civil Engrs., Vol. 100, p. 1361, 1935.)*

The plotting of a flow net is at best a rather tedious process of cut and try. Where the boundaries are known as in the case of flow through a homogeneous soil between the impervious base of a dam and an impervious stratum in the foundation, the construction of the flow net is relatively simple (see Fig. 9).

For a flow net through an earth dam and its foundation it is well to first compute the position of the seepage line as in Arts. 10 and 14 or possibly by some other more approximate means, as in Art. 16. One will then have the upper boundary. The seepage line is also a "flow line" of the flow net and accordingly the equipotential lines will intersect it at right angles and also at equal vertical intervals. (See Art. 14, Chapter 3.) The lower boundary which may be an impervious foundation or an impervious stratum in the foundation will also be known or if not known should be determined as nearly as practicable. In Figs. 10 and 11 are shown more or less typical flow nets through earth dams on impervious foundations.

Arthur Casagrande [9] gives some excellent hints to the engineer who would construct a flow net.

> 1. Use every opportunity to study the appearance of well-constructed flow nets. When the picture is sufficiently absorbed in your mind, try to draw the same flow net without looking at the available solution; repeat this until you are able to sketch this flow net in a satisfactory manner.

[9] ARTHUR CASAGRANDE, "Seepage Through Dams," *J. New Eng. Water Works Assoc.*, June, 1937, pp. 136 and 137.

2. Four or five flow channels are usually sufficient for the first attempts; the use of too many flow channels may distract the attention from the essential features.

3. Always watch the appearance of the entire flow net. Do not try to adjust details before the entire flow net is approximately correct.

4. Frequently there are portions of a flow net in which the flow lines should be approximately straight and parallel lines. The flow channels are then about of equal width and the squares are therefore uniform in size. By starting to plot the flow net in such an area, assuming it to consist of straight lines, one can facilitate the solution.

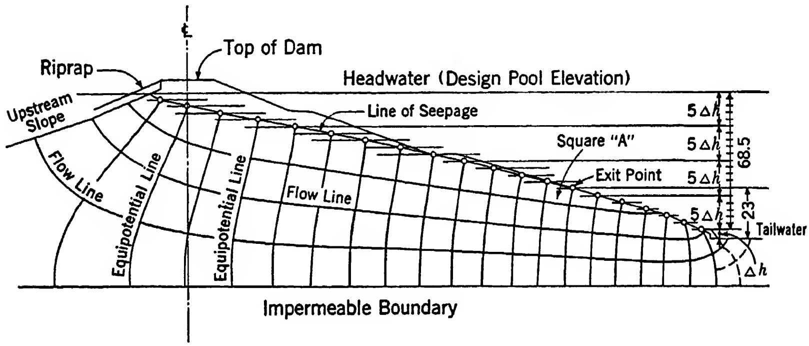


FIG. 10. Flow net for dam of homogeneous material and flat downstream slope. (*Prepared by Ralph Hansen, Soils Engineer, Little Rock District, U. S. Army Engineers.*)

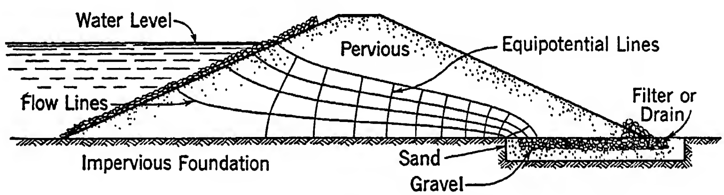


FIG. 11. Flow net for dam of homogeneous material but with a drainage system.

5. The flow net in confined areas, limited by parallel boundaries, is frequently symmetrical, consisting of curves of elliptical shape.

6. The beginner usually makes the mistake of drawing too sharp transitions between the straight and curved sections of flow lines or equipotential lines. Keep in mind that all transitions are smooth, of elliptical or parabolic shape. The size of the squares in each channel will change gradually.

7. In general, the first assumption of flow channels will not result in a flow net consisting throughout of squares. The drop in head between neighbouring equipotential lines corresponding to the arbitrary number of flow channels will usually not be an integer of the total drop in head. Thus, where the flow net is ended, a row of rectangles will remain. For usual purposes this has no disadvantages, and the last row is taken into consideration in computations by estimating the ratio of the sides of the rectangles. If for the sake of appearance, it is desired to resolve the entire area into squares, then it becomes necessary to change the number of flow channels, either by interpolation or by a new start. One should not attempt to force the change into squares by adjustments in the neighbouring areas, unless the necessary correction is very small.

8. Boundary conditions may introduce singularities into the flow net.
9. A discharge face, in contact with air, is neither a flow line nor an equipotential line. Therefore, the squares along such a boundary are incomplete. However, such a boundary must fulfill the same condition as the line of seepage regarding equal drops in head between the points where the equipotential lines intersect.

13. Hydraulic Electric Analogy. The fact that the equation for Ohm's law is of the same form as the Darcy equation is responsible for the Hydraulic Electric Analogy (see Art. 21, Eq. 5, Chapter 16). Ohm's law is

$$I = \frac{E}{R} \qquad [11]$$

where I = current (quantity of electricity per unit of time),
E = potential (pressure; head),
R = resistance.

Harza [10] describes his electric analogy tray for tracing out a flow net between two known impervious boundaries as follows:

The writer's tray is connected somewhat differently from that of previous investigators, as shown in Fig. 12. It is 24 by 46 in. in size, utilizing a salt

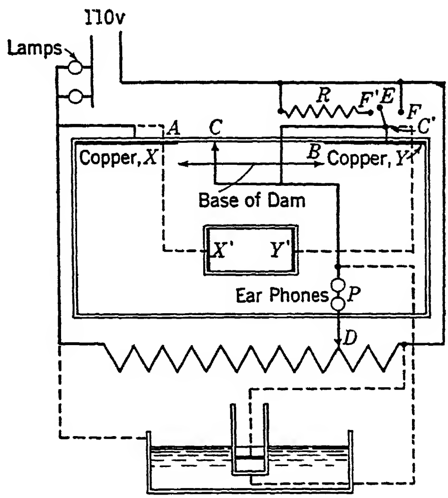


FIG. 12. Arrangement of Harza electric analogy tray. (*From "Uplift and Seepage Under Dams on Sand," by L. F. Harza, Trans. Am. Soc. Civil Engrs., 1935.*)

solution about 1 in. deep. Copper terminals, X and Y, permit simulating the base of a dam, 20 in. or less in width, A, B, of any desired sectional design. With switch E closed to F, the earphones, P, permit the determination of the

[10] From L. F. HARZA, "Uplift and Seepage Under Dams on Sand," *Trans. Am. Soc. Civil Engrs.*, Vol. 100, 1935, pp. 1362 and 1363.

proportionate voltage (or hydrostatic uplift head) drop at C along the base by selecting point D to produce silence. Likewise, to determine the resistance of the solution between the proper strips, X and Y, Switch E is closed to F' through the known resistance, R, with earphones connected to C'.

To obtain unit resistance of the solution for parallel flow the two main terminals are changed from the copper strips, X and Y, to X' and Y', in a bakelite frame temporarily immersed; or the copper strips, X and Y, are replaced by strips along the entire ends of the tray while the model of the dam base is removed. Knowledge of the exact depth of the solution is not necessary and would be difficult to determine in such a shallow tray and likewise would be subject to frequent change by evaporation and replacement. It is only necessary to know the resistance between the terminals, X and Y, compared with the resistance producing parallel flow, as between X' and Y'. in an equal depth of solution.

The hydraulic-electric analogy tray is particularly useful in constructing flow nets for cross-sections of dams which are not composed of homogeneous materials.

Thus a dam might be composed of a central section for which the value of K was, say, 1/20th that of the shells. Then for the model in the electric analogy tray, the unit electric resistance of the central section would be made 20 times that of the shells. This may be accomplished by having the electrolyte over the central section of the tray model 1/20th of the depth of that over the shell section.

14. Seepage Line in Earth Dam of Composite Cross-Section. An earth dam which consists of a central section of highly impervious material, as a silty clay,

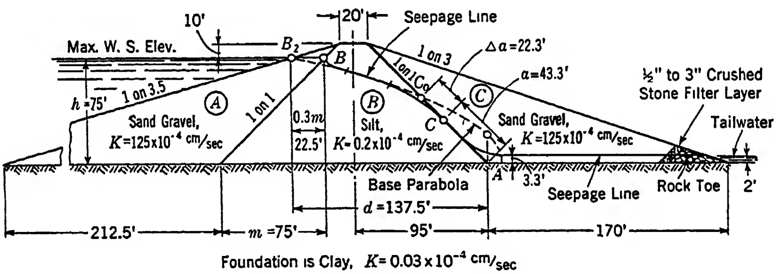


FIG. 13. Determination of seepage line in dam of composite section.

with shells of very pervious material is frequently a very desirable type of dam both from the standpoint of stability and watertightness provided that the dam is on a firm and relatively impervious foundation. The cross-section of such a dam is shown in Fig. 13.

The shells of sand gravel are doubtless several hundred times as pervious as the central portion of the dam consisting of clayey silt. The upstream pervious shell will have practically no effect on the position of the phreatic line, and the downstream pervious shell will act as a drain. Owing to the tremendous difference in permeability, the shells will have practically no influence on the position of the seepage line in the central section. Consequently the position of the

seepage line may be determined for the central section by the methods discussed in Art. 10.

In the downstream pervious shell of the dam the seepage line will rise above tailwater only slightly, just enough to provide the necessary head for the slight amount of water which gets through the central impervious section to flow out through the downstream shell. The approximate position of the seepage line for such a section is shown in Fig. 13.

It is seldom necessary to determine by precise methods the seepage line for the entire cross-section of a composite dam provided that the position of this line is carefully determined for the most impervious portion of the dam. Sometimes when the permeability coefficients of the various sections of the dam are fairly close together, it may be desirable to get the position of the seepage line throughout the entire cross-section. In making such computations it is important to bear in mind that at any given point in the section, discharge is the same. The theoretical point of intersection of the seepage line with the boundary is assumed and changed until the computed flow through each section comes out the same.

15. Computed Position of the Seepage Line in Composite Section. As an example we will consider the composite earth dam whose cross-section is shown in Fig. 13. It will be noted that the foundation consists of a relatively impervious clay soil. The upstream portion and the downstream portion of the dam consist of a natural mixture of sand gravel which is quite pervious while the central section of the dam consists of a relatively impervious silt.

Example. First construct the base parabola, using Eq. 2. The value of y_0 is determined from Eq. 3, so that with $h = 75.0$ and $d = 137.5$, $y_0 = 19.2$. $m = 75.0$.

Substituting the value of y_0 in Eq. 6, we may solve for values of y for corresponding assumed values of x, and from these values of x and y the base parabola may be plotted. (See Fig. 6.)

When $x = d$, $y = h$, so that the base parabola will pass through point B_2 located $0.3m$(22.5 ft) upstream from the point of intersection of the free water surface with the upstream face of the more impervious section, as shown in Fig. 13.

The intersection of the base parabola with the downstream face at C_0 is found from Eq. 5, the polar equation of the base parabola.

$$a + \Delta a = \frac{19.2}{1 - 0.707} = 65.6 \text{ ft}$$

From Fig. 7*b*, the value of Δa at $\alpha = 45°$ is 0.34, $(a + \Delta a) = 0.34 \times 65.6 =$ 22.3 ft. Point C_0 is therefore $65.6 - 22.3 = 43.3$ ft, measured up along the downstream face from point A. The seepage line is completed at the downstream end by sketching a short transition curve from point C to the base parabola.

The upstream end of the seepage line is sketched in with a short transition curve as indicated in Fig. 13.

The seepage flow per unit of dam width may now be determined from Eq. 8

where $q = k(\sqrt{d^2 + h^2} - d)$,
$k = 0.2 \times 10^{-4}$ cm/sec $= 0.4 \times 10^{-4}$/ft/min,
$q = 0.00004(\sqrt{137.5^2 + 75^2} - 137.5)$,
$q = 0.00076$ cu ft/min.

This will, of course, also be the flow through the downstream pervious section of the dam, Fig. 13, which has a permeability coefficient of 125×10^{-4} cm/sec (0.025 per ft per min).

In Fig. 13, the seepage line at the toe of the downstream shell is shown as 2 ft above the impervious foundation level due to tailwater. The Darcy formula may be used to determine the approximate position of the seepage line in the downstream shell as follows:

Let A' represent the height of the seepage line at the toe of the shell ($=2$ ft), h' difference in elevation of the seepage line within the shell, and l the length of path in the shell ($=110$ ft), and k' the permeability coefficient for the shell ($=125 \times 10^{-4}$ ft/min). Thus

$$q = k'iA = k'\frac{h'}{l}\left(A' + \frac{h'}{2}\right) \tag{12}$$

where q is the same as the seepage through the core ($=0.00076$ cu ft/min), $A' + h'/2$ is the average area of cross-section, and h'/l is the average gradient.

$$0.00076 = 0.025 \times \frac{h'}{110}\left(2 + \frac{h'}{2}\right)$$

$$h'^2 + 4h' = 6.7$$

$$h' = 1.27 \text{ ft, use } 1.3 \text{ ft}$$

Thus, the height of the seepage line is 1.3 + 2 or 3.3 ft above the impervious foundation at the upstream end of the shell.

We may now complete the seepage line for Fig. 13 with a sufficient degree of precision. The upstream pervious section is so excessively pervious as compared with the central section that it will have no practical effect on the seepage line. Hence, through this section it is drawn as a straight line in Fig. 13.

At the entrance to the relatively impervious central section of the dam the seepage line will enter approximately perpendicular to the slope but will promptly curve over to join the theoretical seepage parabola, determined as already indicated. When the seepage line issues from the downstream discharge face of the central impervious section, it will tend to continue very nearly in the same direction as the slope of the discharge face unless affected by tailwater. It is here assumed that the downstream section is relatively pervious, as indicated in Fig. 13. A satisfactory method of determining the position of the seepage line through the downstream section under the given condition of Fig. 13 has been given, and the transition from the point of egress to the seepage line for the impervious central section may readily be drawn in as a transition curve.

16. Rough Method of Predicting Position of Seepage Line. The development of methods for locating the position of the seepage lines in the cross-section of earth dams has proved an irresistible loadstone to many soil mechanicians with the result that there are hundreds of pages of text (some of it very mathematical) dealing with the subject. A study of the actual line of seepage in various earth dams (see Figs. 16 to 19) shows that precision in prediction of the line of seepage is not obtainable. In the opinion of the authors, methods more precise than the ones already given are not justified and in many cases much rougher methods are suitable.

Referring to Fig. 6, it is shown herein that the vertical distance, e, which is the distance from the impervious base of the dam up to the intersection of the seepage line, is equal to approximately h 3.

In Fig. 6 the discharge through the dam may be expressed by the equation

$$q = \frac{k(h - e)}{l} \frac{(h + e)}{2} = \frac{k}{2} \frac{(h^2 - e^2)}{l} \qquad [13]$$

where l = mean length of seepage path (see Fig. 14),

$$l = 2(h + Z) \cot\alpha + w - 0.7h \cot\alpha - \frac{e}{2} \cot\alpha$$

$$= \left(1.3h + 2Z - \frac{e}{2}\right) \cot\alpha + w$$

in which Z is vertical distance from headwater to the top of the dam and w is the top width of the dam.

The above is simply Darcy's law (Eq. 5, Chapter 16) applied to unit width of dam and assuming that the mean discharge area will be $(h + e)/2$. The equation agrees closely with Eq. 9 for relatively flat slopes, indicating about 10 per cent greater values for slopes as steep as 1 on 1 ($\alpha = 45°$).

The value of e for which q is maximum may be determined by trial and error or by differentiation and is approximately equal to $h/3$ for discharge slopes flatter than about 1 on 1, thus giving the position of C, the point of intersection of the seepage tangent with the discharge face. Changes in the value of k do not produce any change in this relationship.

Substituting $h/3$ for e in Eq. 13, we obtain

$$q = k \frac{(h^2 - h^2/9)}{2l} = \frac{4kh^2}{9l} \qquad [14]$$

Thus for Fig. 14 the rough method would give $e = h/3 = 75/3 = 25$ ft. For convenience the central portion B of Fig. 13 is replotted in Fig. 14. Using e = 25 ft, C (point of intersection of seepage line and discharge face) is then plotted on Fig. 14. Next starting from a point on maximum water surface = $0.3m$ upstream from the face (here = 22.5 ft) draw the seepage tangent to point C on Fig. 14.

Finally to get the approximate actual seepage line, draw in the ingress line with its perpendicular and curved line joining onto the seepage tangent, as shown

in Fig. 14 and described in Art. 15. The mean length of path of seepage, from Fig. 14, is

$$l = \left(1.3 \times 75 + 2 \times 10 - \frac{25}{2}\right) \times 1 + 20 = 125 \text{ ft}$$

The discharge through the dam per foot of width may be computed by Eq. 14 as follows:

$$q = \frac{4 \times .00004 \times 75^2}{9 \times 125}$$

q = 0.00080 cu ft per min per lin ft of dam. This compares with q = 0.00076 cu ft per min per lin ft as computed for the same dam by Eq. 8 of Art. 10.

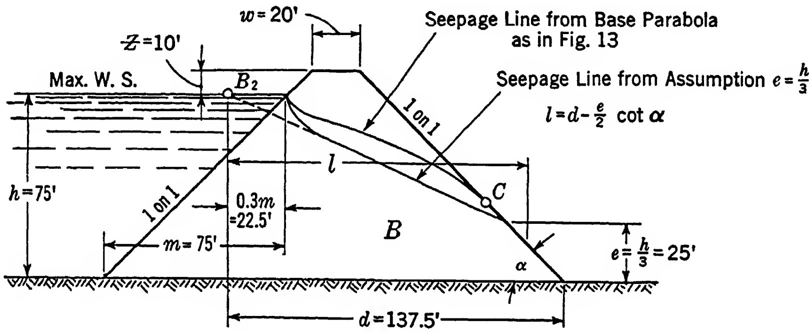


FIG. 14. Central portion (B) of Fig. 13, showing comparison of rough method of plotting seepage line (see Art. 16) with seepage parabola.

These two values of discharge agree more closely than it is possible to estimate the average permeability coefficient for an actual dam; therefore the suggested approximate method of locating the seepage line and computing the seepage is good enough for all practical purposes.

17. Location of Seepage Line Where Foundation is Pervious. If, instead of the impervious foundation as has been so far assumed, a considerable layer of relatively pervious material overlies the rock or an impervious layer, the location of the seepage line may be obtained by the methods already described by assuming that the foundation is still a boundary; but below this boundary discharge takes place through the pervious stratum down to the impervious stratum in the foundation.

Thus in Fig. 15 all the conditions and dimensions are the same as in Fig. 13 except that in Fig. 15*b* the foundation of the dam is as permeable as the core down to a depth of 40 ft below the base of the dam. The position of the seepage line has been determined in Fig. 15 by the methods outlined in Art. 10. It will be noted that the position of the seepage line is identical with that in Fig. 15*a*, where the foundation was impervious.

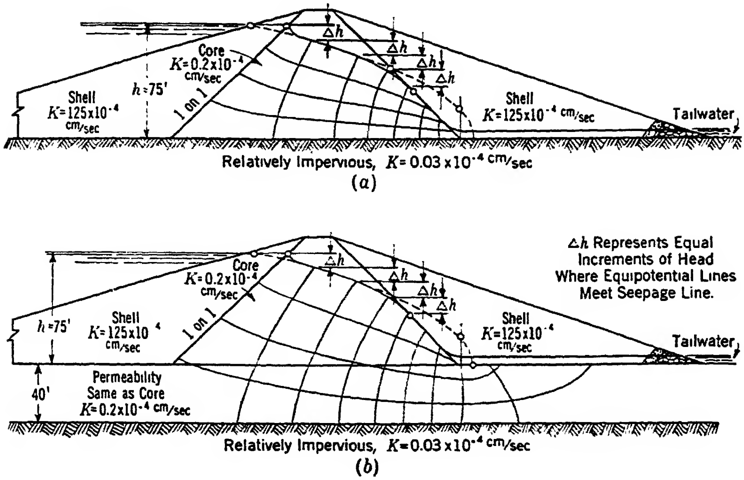


FIG. 15. Location of seepage line where (a) Foundation is same as in Fig. 13 except flow net is sketched in. (b) Seepage line is same as in Fig. 13, but pervious stratum under dam.

18. Seepage Line in Existing Earth Dams. The position of the seepage line shown in Fig. 28 for the glass tank model of the dam is in substantial accord with the position of the seepage line as determined by the methods discussed in Arts. 10 to 17. Figs. 16 to 19 show the actual positions of the seepage line in several

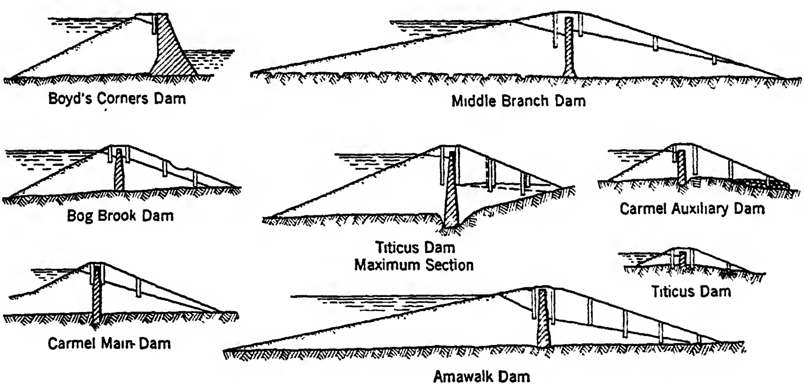


FIG. 16. Actual seepage lines in earth dams. (*From Caleb Mills Saville, Trans. Am. Soc. Civil Engrs., p. 98, Vol. 87, 1924.*)

earth dams. Many of these cross-sections are interesting, but, in general, the information with regard to the materials in the dam is fragmentary and when available is more qualitative than quantitative.

The section of the Minatare Dam, Fig. 18*d*, is interesting as showing the effect of a cutoff and special drainage measures in the position of the seepage line. The

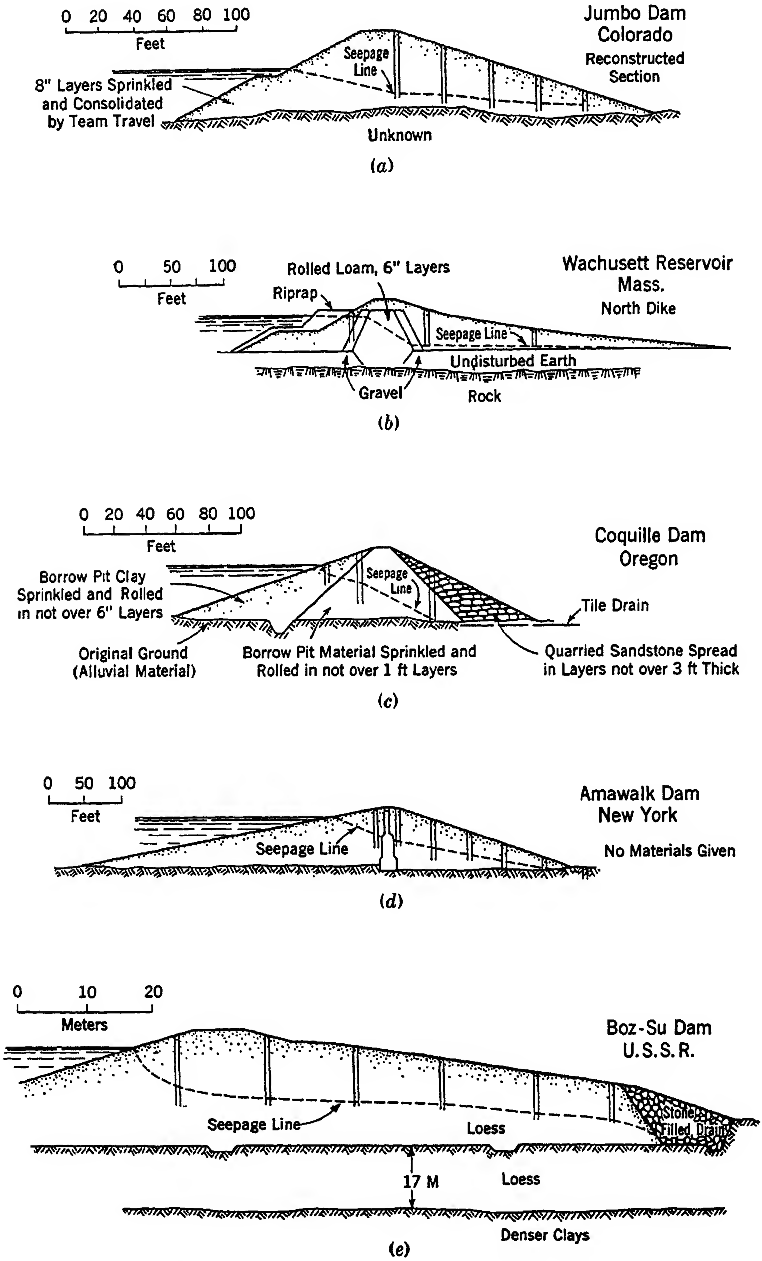


FIG. 17. Actual seepage lines in earth dams. (*Data from R. C. Haven, Tech. Mem. 389 U. S. Bur. Reclamation.*)

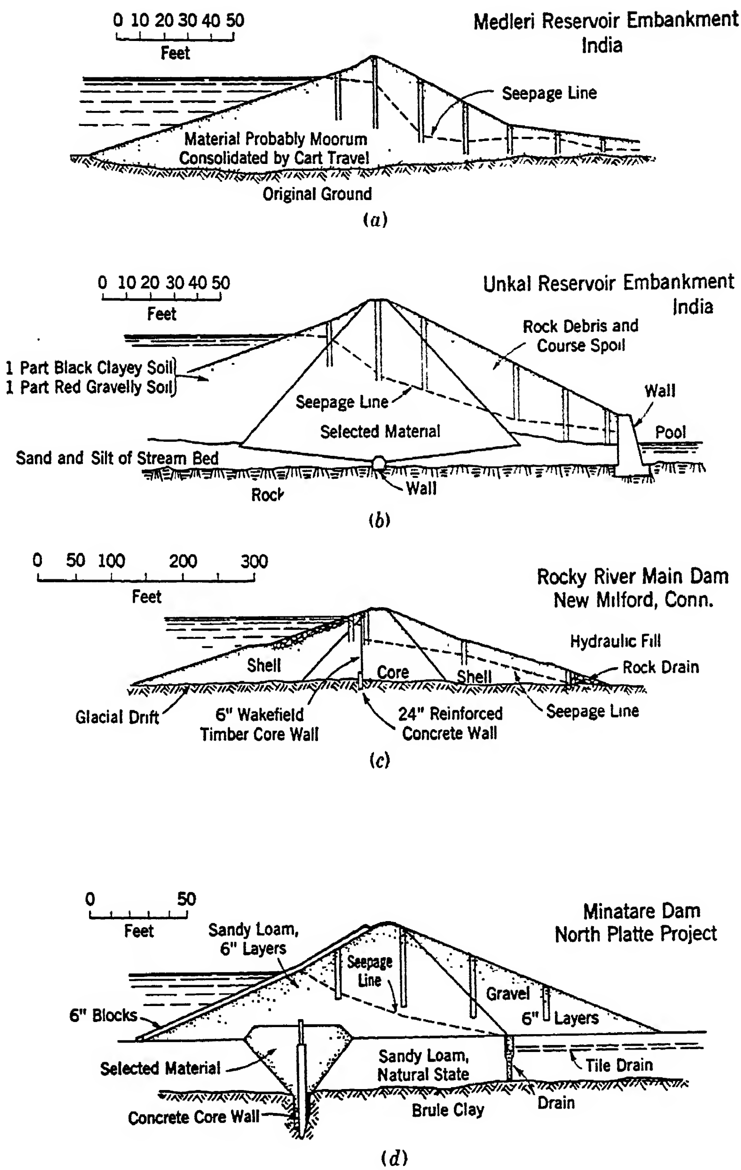


FIG. 18. Actual seepage lines in earth dams. (*Data from R. C. Haven, Tech. Mem 389 U. S. Bur. Reclamation.*)

effect of a core wall in lowering the line of seepage in a highly pervious dam on a highly impervious base is shown in the case of Wissota, Fig. 19*a*.

Of the 20 earth dams for which seepage lines are shown, 9 are from the records gathered by the U. S. Bureau of Reclamation, Technical Memoranda Numbers 389 and 493 by R. C. Haven. These pamphlets show the seepage lines for a

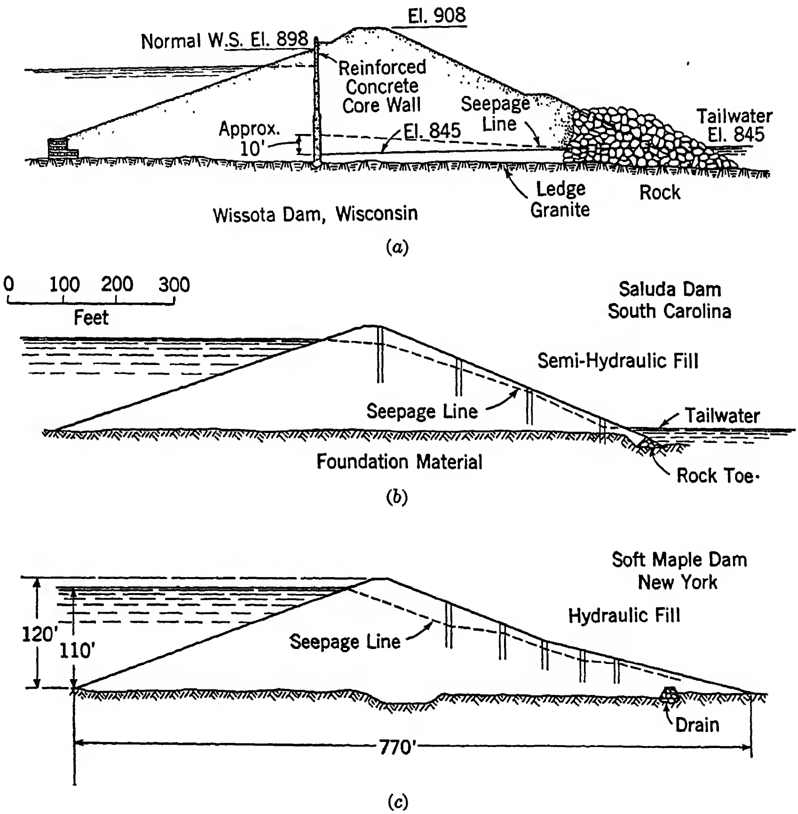


FIG. 19. Actual seepage lines in earth dams.

considerable number of earth dams, together with some information as to the materials in the dams and in some cases additional information on construction and foundation conditions.

19. Quantity of Seepage. All methods of computing seepage under or through earth dams are primarily based on Darcy's law: $q = kiA$, Eq. 5, Art. 21, Chapter 16. Of the following formulas for seepage, three have already been developed in connection with the determination of the seepage line in the foregoing sections, but they are assembled here to present a résumé of formulas for computing the seepage through earth dams.

In all these formulas the symbols have the following meaning:

q = discharge in cubic feet per minute per foot of width,
k = permeability coefficient in feet per minute,
i = hydraulic gradient,
h = head causing seepage,
l = length of path of seepage in which loss of head h takes place; thus
$i = h/l$,
d = horizontal distance from the point where seepage tangent intersects the maximum headwater surface to the downstream toe of the section as in Fig. 6,
α = angle which discharge face makes with the horizontal (Fig. 6),
e = vertical distance from base to point where the seepage line intersects the discharge face,
a = length of line on the discharge face from downstream toe up to point of intersection with seepage line (see Fig. 6).

$$q = ka \sin^2 \alpha \qquad [9] \quad \text{(See Fig. 6 and Art. 10)}$$

$$\text{where } a = S_0 - \sqrt{S_0^2 - \frac{h^2}{\sin^2 \alpha}} = \sqrt{h^2 + d^2} - \sqrt{d^2 - h^2 \cot^2 \alpha}$$

$S_0 = \sqrt{h^2 + d^2}$, applicable where α is less than 30°.

$$q = k(\sqrt{d^2 + h^2} - d \qquad [8] \quad \text{(See Fig. 6 and Art. 10)}$$

applicable for α between 30° and 180°.

$$q = \frac{4kh^2}{9l} \qquad [14] \quad \text{(Art. 16)}$$

where $l = (1.3h + 2Z - e/2) \cot \alpha + w$,
$= (1.133h + 2Z) \cot \alpha + w$, since $e = h/3$,
Z = freeboard,
w = top width, see Fig. 14.

The above is an approximate equation, applicable for α less than 90°.

For a section where the horizontal and vertical permeability coefficients differ materially, the value of k used in the above equations is determined as follows:

$$k = \sqrt{k_v \times k_h} \qquad [15]$$

where k_v is the vertical permeability coefficient and k_h is the horizontal permeability.[11]

While usually in natural formations the horizontal permeability is from 4 to 10 or more times the vertical, there are exceptions. Thus loess in its natural undisturbed condition is sometimes 20 to 50 times as pervious in a vertical direction as in a horizontal direction. This is because of the vertical passageways or tubercles which abound in loess formations.

[11] See ARTHUR CASAGRANDE, "Seepage Through Dams," *J. New Eng. Water Works Assoc.*, June 1937, p. 152.

20. Total Seepage Through and Under Earth Dams. If the base of the dam is pervious and if it is desired to determine the total seepage through and under the dam, proceed as in the following example. Referring to Fig. 15 and Art. 17

$$q_T = q_D + q_F \tag{16}$$

(see Fig. 15 and Art. 19) where q_T is total discharge through the dam and foundation per foot of width and q_D is discharge per foot of width through the dam, which for the case illustrated in Art. 15 was equal to 0.00076 cu ft per min per ft of width, and q_F is discharge through the foundation per foot of width.

(Note that in Fig. 13 $q_T = q_D$ since the foundation is impervious.)

In Fig. 15, using the Darcy equation

$$q_F = k_F \frac{h}{l} A \tag{5}$$

$k_F = 0.2 \times 10^{-4}$ cm/sec $= 0.4 \times 10^{-4}$ cu ft per min $= 0.00004$ cu ft/min,
$h = 75.0$, $l = 190$ (see Fig. 14), $A = 40$,

$$q_F = \frac{0.00004 \times 75 \times 40}{190} = 0.00063 \text{ cu ft/min},$$

$q_D = 0.00076$ cu ft per min as in Art. 15,
$q_T = 0.00076 + 0.00063 = 0.00139$ cu ft per min per lin ft

Where a composite dam consisting of a rather impervious core and pervious shells rests on a foundation through which the seepage is appreciable, the foundation seepage is likely to appear in the downstream shell and should be accounted for in drawing the seepage line through the shell as described in Art. 15.

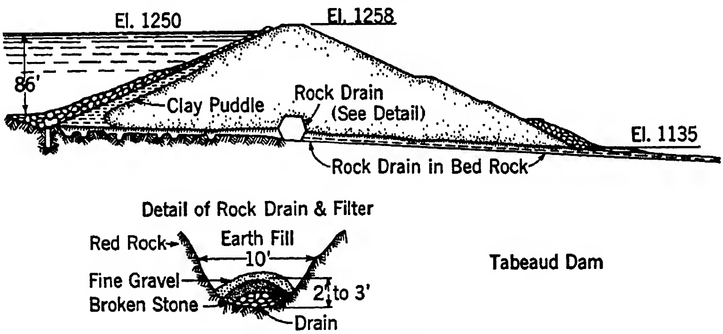


FIG. 20. Tabeaud Dam, California, showing drainage measures adopted (1902).

21. Drainage in Earth Dams. Although some soils engineers would have us believe that drainage and filters in earth dams began with the present school of soils mechanics the use of such measures is not new and has been understood by some engineers for many years.

For instance, there is the Tabeaud Dam [12] (Fig. 20) in California constructed in 1902 having a maximum height of 123 ft. The maximum section was short as

[12] See BURR BASSELL, *Earth Dams*, p. 10, 1907, published by Eng. News Publishing Co., N. Y.

the base was merely the bottom of a relatively narrow canyon. A drainage ditch was cut in the rock foundation at the maximum cross-section and was carried upstream to approximately the center line of the dam, at which point similar longitudinal drains were installed for practically the entire length of the dam. Fig. 20 shows a section of the Tabeaud Dam indicating the position of the drains, together with a typical cross-section of the drain.

The protection of the drain by a filter should be particularly noted. The large stones in the bottom of the drain were placed so as to give a definite continuous opening along the bottom of the rock trench. The drain was then surrounded with 18 in. of crushed stone 3 in. down to 1 in., over which was placed fine gravel. Above this gravel the trench was refilled with gravelly clay.

The Tytam Bay Dam, Hong Kong,[13] built in 1907, included a rock-fill drain which extended upstream to the center of the dam, where there was a concrete core wall. At Wissota Dam, Wisconsin,[14] built in 1916, 6-in. terra cotta pipe and a layer of loose rock and gravel surrounded by sand were utilized in the downstream portion to promote drainage and to lower the seepage line. (See Fig. 19*a*.)

In 1902 Frederic P. Sterns remarked, "Few engineers who deal with filtration would wish to say that they could not design the downstream portion of such a dam in a way to permit the water filtering through the dam to come to the surface without carrying earth with it." [15]

Bassell (1907) [16] stated that thorough drainage of the base of a dam is a matter of vital necessity.

Many other earth dams constructed before 1920 had extensive upstream and downstream fills of gravel and/or rock for the purpose of improving both drainage and stability. The Ashokan Dikes of the New York Board of Water Supply (Fig. 32) was, it will be noted, provided with such fills of loose rock on both faces.

For a dam such as that indicated in Fig. 5, relatively impervious and homogeneous, it is evident that the seepage line would intersect the downstream face well above the toe as in Fig. 6 unless some method of drainage is adopted. In Fig. 5, drainage is accomplished and the seepage line lowered to a point well within the downstream face by the filter layer at the base of the dam.

In some dams drainage is no problem at all because the material in the downstream portion is so pervious that drainage conditions are already excellent. Thus a design like that shown in Fig. 3 with its pervious shells, under the given conditions, provides ample drainage. Even here some attention must be given to the protection of the rock-fill toe. If the rock, say from spalls to several cubic feet in size, were merely dumped at the toe on the consolidated clay of the foundation and not protected at all, in time it would become impregnated with

[13] JOEL D. JUSTIN, *Earth Dam Projects*, p. 231. John Wiley & Sons, N. Y., 1932.

[14] *Earth Dam Projects*, p. 322.

[15] *Trans. Am. Soc. Civil Engrs.*, Vol. 48, p. 279. Experiments in connection with the design and construction of Wachusett Dikes.

[16] BURR BASSELL, op. cit.

finer material and its efficiency would be seriously impaired or destroyed. Consequently, a filter (see Art. 22) is placed around that portion of the rock toe which would be in contact with the rest of the dam embankment and also with the consolidated clay foundation. This latter point is frequently neglected, but the authors believe that it is frequently advisable to use a horizontal filter layer on top of the impervious foundation because any possible upward movement of water from the foundation might eventually fill the large voids in the rock fill with fine clay or silt and thus ruin the usefulness of the rock toe and cause the seepage line to come out of the downstream face above the rock toe, with possible resulting sloughing.

In Fig. 21 is shown a cross-section of the Arkabutla Dam (1941). It will be noted that the drainage layer (or filter layer) extends along the base in much the

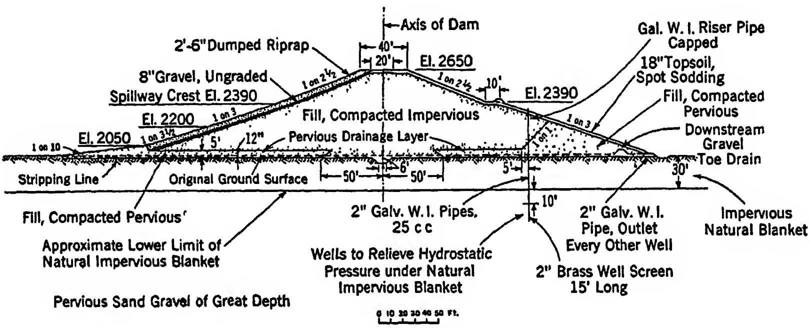


FIG. 21. Valley section Arkabutla Dam, Mississippi, showing drainage layer at base.

same manner as in Fig. 5, except that the drainage (or filter) layer is divided into two parts, upstream and downstream. The upstream portion is of use only in connection with the consolidation of the structure and foundation under its own load. At Arkabutla the foundation consists of a stratum of alluvial clay, which is not fully consolidated. About 30 ft beneath this there is a sand gravel stratum of great depth. Consequently, the drainage layer and the downstream pervious section had two functions to perform as follows:

1. To facilitate and hasten the consolidation of the foundation stratum of clay as the load of the dam was applied during construction, water being squeezed out of the clay layer.

2. To lower the line of seepage so that there was no chance for the downstream face to become saturated. As pointed out in Art. 10, a homogeneous cross-section will have the lower portion of the downstream face saturated (very likely causing sloughs) unless special drainage measures are adopted.

A somewhat unusual drainage feature of the Arkabutla Dam is shown in Fig. 21. The water in the sand and gravel foundation strata below the natural impervious blanket is under some artesian pressure. Hence, at a point about midway between toe and axis line, well points were driven into the sand gravel stratum. At the surface the well points (25 ft center to center) were connected laterally with egress to the toe at 50-ft intervals.

At Sardis, where the impervious natural blanket was only about 10 ft thick, the same problem of foundation drainage was handled by cutting through the impervious layer to the sand gravel layer just downstream from the impervious central core. (See Fig. 23, Chapter 19.)

The Mohawk Dam (1935), a typical cross-section of which is shown in Fig. 22, is an illustration of a type of dam where excellent drainage conditions are assured by the use of a heavy downstream fill of loose rock. The foundation of this dam is quite pervious, the average permeability coefficient of the foundation being about 300×10^{-4} cm per sec. Extending approximately 625 ft upstream from the center line, an impervious natural and artificial blanket insured a long path of percolation for the water seeping under the dam. It will be noted in Fig. 22

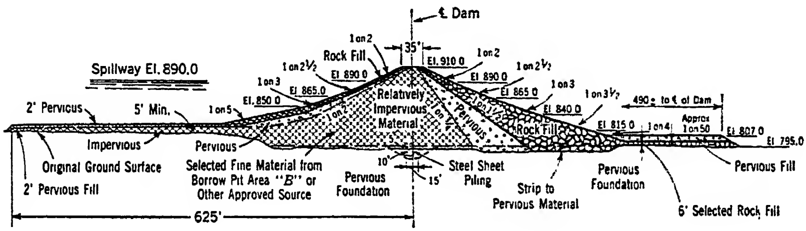


FIG. 22. Mohawk Dam, Ohio (1935).

that the more or less impervious top soil was stripped off downstream from the impervious section of the dam, thus permitting the seepage water passing under the dam to escape into the pervious downstream section of the dam and get away without causing any trouble.

22. Effect of Drainage on Line of Seepage. In Fig. 23 it will be noted that the dam is composed of a homogeneous embankment material on a foundation of

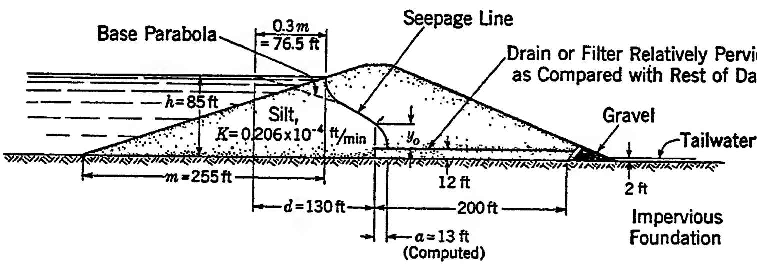


FIG. 23. Effect of drainage on seepage line in homogeneous section as computed in Art. 22.

impervious material but that downstream from the center line there is a drainage layer or filter along the base which discharges through a rock toe fill to the downstream toe of the dam. The upstream end of the filter corresponds to the toe of a dam where $\alpha = 180°$.

Referring to Fig. 7b, $\Delta a = 0$.

Thus, as shown in Fig. 7*a*, for $\alpha = 180°$, $a = a_0 = y_0/2$.

$$y_0 = \sqrt{h^2 + d^2} - d \qquad [3]$$

$$a = \tfrac{1}{2}(\sqrt{85^2 + 130^2} - 130)$$

$$a = \tfrac{26}{2} = 13 \text{ ft}$$

which means that the seepage line will reach the filter layer at a point 13 ft from its upstream end and from the above $y_0 = 2 \times 13 = 26$ ft, which gives us a second point on the seepage line. Another point is obtained by measuring upstream along maximum water surface a distance equal to $0.3m$ upstream from the point where the maximum water surface intersects the upstream face, m being the horizontal projection of the wetted upstream face.

The seepage line may now be drawn in accordance with the principles described in Art. 10. It will be noted that the downstream portion of the dam is practically free from saturation. This is entirely due to the drainage or filter layer which pulls down the seepage line and conducts the seepage to the rock toe. Except for this drainage means, the seepage line would have shown a position similar to that in Fig. 6.

The capacity of the filter should materially exceed the maximum discharge through the dam. By the use of Darcy's formula (Eq. 5, Chapter 16) one may check the capacity of the drainage or filter layer. If the capacity of this filter layer is less than twice the estimated discharge through the dam (by Eqs. 8, 9, or 14), measures should be taken to increase the capacity by enlargement of filter layer, use of more pervious materials, use of pipe, etc., provided that the drainage layer is not made so coarse that there is danger of the filter becoming impregnated.

23. Pipe Drains. Pipe drains are sometimes used in earth dams, especially where the material is so extremely pervious that a large quantity of seepage may be expected. Such pipe drains are laid in and are surrounded by a filter of pervious material. Holes are drilled in the pipes of a size such that practically none of the surrounding pervious material can get through the holes.

Short lengths of cast iron pipe (Class C or D, according to superimposed load) make about the best pipe drains. With proper attention to supporting while laying there is practically no danger of breakage, and their life is practically indefinite. Bell and spigot C.I. pipe should be used and joints should be left open or poured with a mastic to permit movement. Holes must be drilled instead of punched, as with steel pipe, which adds to the cost. Class A pipe is too light and should never be used for drains of this type.

Terra cotta sewer pipe is sometimes used for drains, but because of the possibility of breakage due to carelessness in laying, it should not be used in any situation where its breakage might conceivably result in disaster.

Because of the cost of cast iron pipe, steel pipe is frequently utilized, where the reservoir will become muddy and seal the upstream face and foundation before the pipe rusts out.

The use of ordinary black steel pipe is not recommended because of its relatively short life under many conditions. Galvanized rust resisting corrugated steel pipe dipped in asphalt has been extensively and successfully used for drainage in earth dams where the quantity of seepage is large and the surrounding material highly pervious. The holes giving the seepage access to the pipe should be punched before the pipe is galvanized.

Some details of the pipe drain utilized at the Kingsley hydraulic fill dam are shown in Fig. 24 and are typical of several pipe drains which have been used.

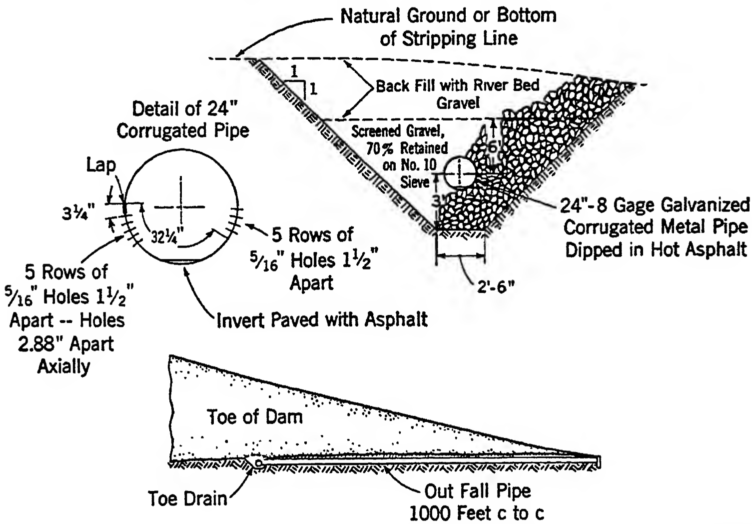


FIG. 24. Galvanized iron asphalt-dipped pipe drain as used at Kingsley Dam, Nebraska

The drain is parallel with the center line of the dam and in the maximum section is located about 240 ft upstream from the downstream toe, where the fill over it is about 60 ft deep. It is placed in a trench excavated in the original ground with the invert about 10 ft below the original surface. At intervals of approximately 1000 ft, there are T connections to an outfall pipe, in most cases 36 in. in diameter. The longitudinal drain is in most cases 24 in. in diameter, with a minimum diameter of 18 in. The large size of outfall was provided not because it was necessary for taking care of the seepage but in order to permit a man to get up to the T and thus be able to clean out the longitudinal drains if that should ever be necessary. The pipes were of corrugated galvanized iron, asphalt dipped.

There is one objection to the use of pipe drains like those just described. They may be separated or broken under conditions such that they act as sluice pipes for carrying away large quantities of the embankment material of which the dam is composed. The writer knows of several cases where this has happened, but the situation was discovered in time to apply remedial measures. For that reason properly filtered rock or gravel drains are generally preferable.

It is possible to protect pipe drains with filters provided the first layers around the pipe are so coarse that if the pipe breaks the stone could not pass through the pipe. Additional filter layers are then necessary around the stone or gravel. They would then be entirely safe against the contingency just discussed. Unfortunately, however, this would take away the principal advantage of the pipe drains, which is that of economy.

The pipe drains are in general used only where the rock drains with their filters are not available except at prohibitive cost. At both Sardis Dam in Mississippi (1940) and Kingsley Dam in Nebraska (1941) rock was extremely expensive and unusually pervious foundations indicated that there would be a large amount of seepage to be taken care of. This led to the adoption of the pipe drains. In addition the pipe drains were so located that there was presumably no probability of their collapse resulting in a serious sluicing of embankment material through them.

24. Filters for Earth Dams. About 1900 the late Frederic P. Sterns made experiments [17] on the impregnation of a corase material with a fine material and found that in accordance with already established filter-plant experience such impregnation did not take place with properly graded layers. Later experimenters, including Terzaghi and G. E. Bertram [18] have made experiments along much the same line. Bertram found that "the minimum critical ratio of the 15 per cent size of the filter at the limit of stability is approximately 9." This conclusion applied to material at least 50 per cent compacted.

In other words, if the flow is, say, downward, each successive layer of material may be composed of particles such that for the 15 per cent size (15 per cent smaller than and 85 per cent larger than) the diameter is 9 times that of the 15 per cent size of the layer above. If the above condition prevails and the material is at least 50 per cent compacted, practically no impregnation will take place. If the ratio is much greater than 9, impregnation may occur.

When one realizes that permeability varies approximately as the square of the diameter, it becomes clear that one may go from a fine silt to sizable broken stone in a very few layers. In many cases the required results can be obtained with only one layer.

In Fig. 25 is given an illustrative example of how this ratio works. Thus beginning with a fine silt of which the embankment is composed, and stepping up the diameter of the 15 per cent size with each successive layer by a ratio of 9, we will have:

Layer Number	Thickness of Layer	Diameter of 15% Size	Approximate Permeability Coefficient
1. Embankment	Indefinite	0.01 mm	0.206×10^{-4} ft per min
2. Fine sand	18 in.	0.09 mm	28.0×10^{-4} ft per min
3. Coarse sand	18 in.	0.81 mm	4200×10^{-4} ft per min
4. Gravel	18 in.	7.3 mm	Unknown

[17] See *Trans. Am. Soc. Civil Engrs.*, Vol. 48, p. 267.

[18] See G. E. BERTRAM, "An Experimental Investigation of Protective Filters," Soil Mechanics, Series No. 7, January 1940, publication of Graduate School of Engineering Harvard University.

For the embankment the 15 per cent size has a diameter of 0.01 mm. It is fine silt and is the relatively impervious material of which the bulk of the embankment is formed. Its permeability factor by test is 0.206×10^{-4} ft per min.

The next layer is 18 in. thick, the 15 per cent size has a particle diameter of $(0.01 \times 9) = 0.09$ mm, a very fine sand, and the permeability coefficient is 28×10^{-4} ft per min.

The next layer is 18 inches thick, the 15 per cent size has a particle diameter of $(0.09 \times 9) = 0.81$ mm, which is a coarse sand, and the permeability coefficient is 4200×10^{-4} ft per min.

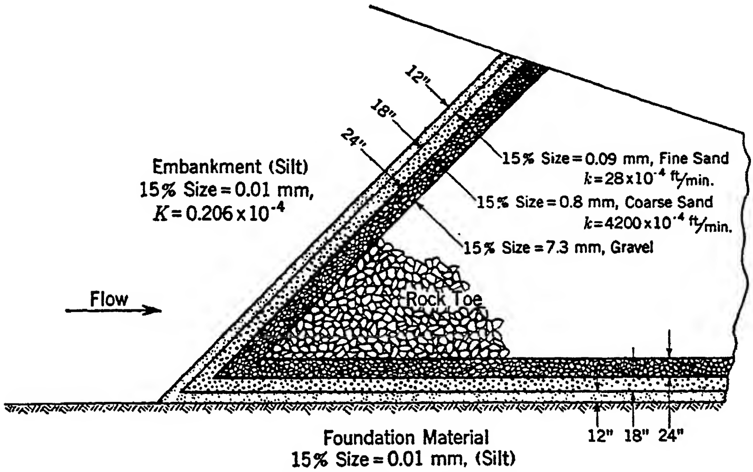


FIG. 25. Filter protecting rock toe with Bertram ratio for particle sizes. (See Art. 24. Note. The 15 per cent size is such that 15 per cent of sample is smaller than, 85 per cent is larger than.)

For the next layer the particle size for the 15 per cent size will be 7.3 mm and the permeability coefficient may be left to the imagination.

In the above the stated coefficients are not exact but are close enough to what would be found by test to be indicative. Theoretically each layer of the filter could be very thin, but practically a reasonable thickness is necessary in order to make sure that some slight readjustment during or after construction does not rupture the layer. As a practical matter the thickness of each filter should be at least 50 times the diameter of the 15 per cent size, and no sand layer should be less than 12 in. thick.

In many cases the importance of using a filter to protect a rock drain or rock toe of a dam is not appreciated. Fortunately if one uses run of quarry rock, including spalls, chips, and dust, a rough filter sometimes results and we then have effective drainage. In some other cases the engineer is insistent on using clean rock, and he uses it for a drain without a filter. Sooner or later such a drain will silt up or become impregnated with fine material.

The author remembers one dam where there was some trouble from sloughing. Deep ditches were dug down the downstream face of the dam and filled with

washed, coarse, crushed stone. In a relatively few years the crushed stone became impregnated with the fine material of the dam and stopped up. Later drains protected by fine gravel and sand were installed, and these functioned properly.

Fig. 26 shows an actual and rather typical rock toe protected by a filter consisting of a layer of sand and a layer of gravel. There was a particular reason for sinking the rock toe into the original surface as indicated. There was a blanket of impervious material about 15 ft thick which formed the immediate foundation of the dam. Below this impervious stratum there was a deep stratum of coarse sand with some gravel under some hydrostatic pressure. It will be noted that the rock toe drain penetrates to this pervious material. There was a

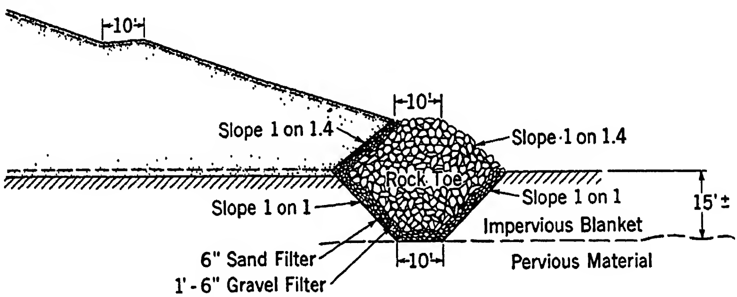


FIG. 26. Typical rock toe with filter protection.

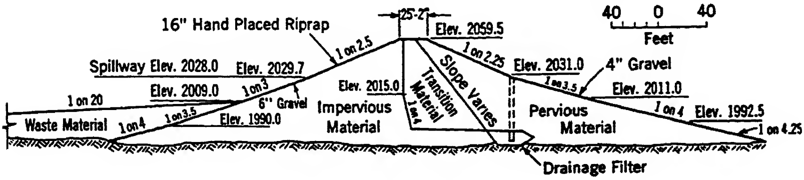
considerable flow from the pervious stratum through the filter to the rock toe drain which discharged to the old river channel.

There are many cases where a single layer of run of bank sand gravel is all that is required to serve both as filter and drain. One should be sure that the sizing of the material will be such that impregnation will be insignificant. The necessary thickness of a proposed filter layer may be readily determined within the necessary degree of precision by the method discussed in Art. 25.

Filters with several layers which must be carefully sized are sometimes necessary. The filter used at Fort Supply Dam (Fig. 27) is illustrative of one of the more elaborate designs, which, because of special conditions, have been considered necessary in some cases. They are sometimes rather expensive and may cost $5 to $8 per cu yd in place.

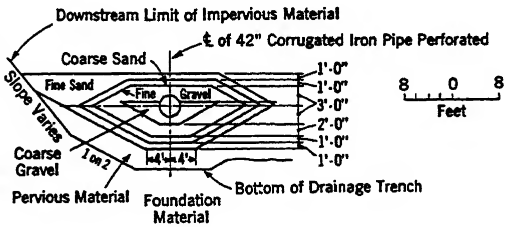
In many cases a run of bank sand gravel may be used successfully, as in Fig. 21. So long as the run of bank material contains the necessary range of sizes, it will make its own filter if thick enough. Fine sand or gravel when placed next to stone will perhaps run right through the interstices in the stone, but the larger gravel will stick and a filter will thus be gradually built up. The first thing to be sure of is that the pervious run of bank materials contains some particles that are of greater diameter than the biggest voids in the stone. Sometimes one can chink the surface of the loose rock drain with spalls until the gravel will hold. A hose under heavy pressure should be used to pack the run of bank gravel into the interstices of the surface of the loose rock.

There have been cases where unjustifiably large expenditures have been made in the use of elaborate and precise filters with a number of carefully sized layers of



Coefficients of Permeability	
Material	Value 10^{-4} cm per sec
Foundation	200
Pervious	20
Impervious	0.0005
Fine Sand	300
Coarse Sand	6,000
Fine Gravel	*30,000
Coarse Gravel	*100,000

*Estimated



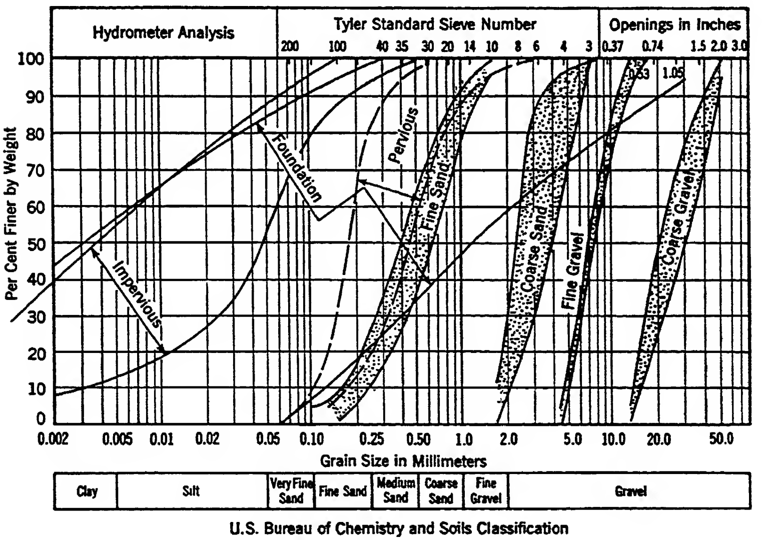


FIG 27. Protective filter details, Fort Supply Dam. (*From Tech. Mem. 175-1, U. S. Waterways Experiment Station.*)

crushed stone and gravel. There are, however, many cases where filters with several layers of different grading are justifiable. Before deciding on elaborate and precise filters of many layers which may cost \$5 to \$8 per cu yd, it is advisable to determine whether it is not more economical to use a much larger quantity of run of bank pervious material which is readily available and cheap. The

problem cannot be determined in the abstract. Each situation should be carefully investigated on its merits.

25. Determination of Required Thickness of Filter Layer. *Example.* As an example take the dam shown in Fig. 23 with its seepage line as computed in Art. 22. From Art. 22, $a = 13$ and $q = ky_0 = 2ka$ (Eq. 7, Art. 10). So far the value of k for Fig. 23 has not been discussed as it was not necessary in order to locate the seepage line provided only that drain was large enough. For the present purposes we will assume that the embankment material is a fine silt and that $k = 0.206 \times 10^{-4} = 0.0000206$ ft per min; hence $q = 0.0000206 \times 13 \times 2 = 0.000536$ cu ft per min.

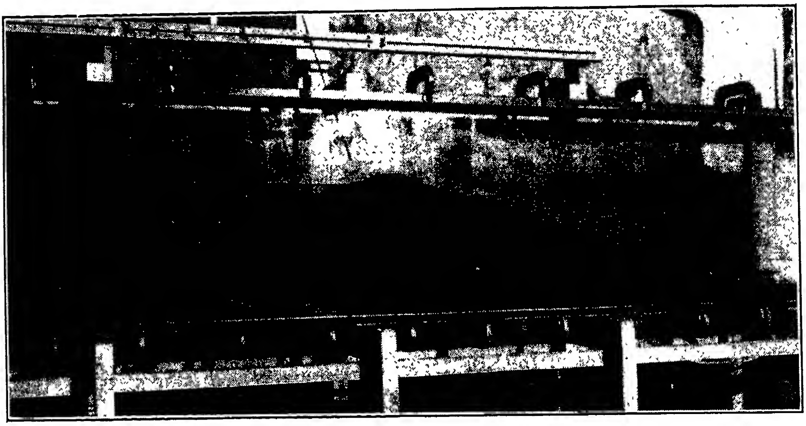


FIG. 28. Seepage line and flow lines in glass tank model of dam. (*Courtesy R. R. Philippe.*)

Now with regard to the drainage layer, we will assume that it is sand containing fine sand and coarse sand. The 20 per cent size is 0.1 mm and the permeability coefficient is found to be 34×10^{-4} ft per min $= 0.0034$ ft per min (see Table 2, Chapter 16). We will also assume that tailwater elevation is 2 ft above the horizontal base of the dam at the rock toe. From Fig. 23 l is 200 ft. Then in the Darcy formula q per unit width $= k\,(h/l)A$ (Eq. 5, Chapter 16).

$$A = \frac{h}{2} + 2$$

Substituting this value of A in the above equation for q we have

$$q = \frac{k}{l}\left(\frac{h^2 + 4h}{2}\right)$$

$$\frac{2ql}{k} = h^2 + 4h$$

$$h^2 + 4h = \frac{2 \times 0.000536 \times 200}{0.0034} = 63.0$$

(Approximate but on the safe side)

$$h = 8.2 - 2 = 6.2$$

and the required thickness of the filter layer is $6.2 + 2 = 8.2$ ft.

To be well on the safe side the filter layer will be made 12 ft thick. If the material in the filter layer were coarser, the required thickness would come out less, but the 15 per cent size of the material in the filter layer here used is about 10 times the diameter of the 15 per cent size of the silt comprising the body of the dam, which is about as large material as could be used without danger of impregnation. More elaborate filters of several layers might be utilized and accomplish the same results, but the quantity of material required for filters and drains would have to work out about 10 per cent of that here used in the filter layer in order to balance out the lower cost of the filter layer shown in Fig. 23.

26. Position of Filters and Drains. Drains and filters for earth dams may be placed in a number of different positions, each position having both advantages and disadvantages. For usual conditions there is no better drain than a generous rock toe about 25 to 35 per cent of the height of the dam with a proper filter to prevent impregnation by fine materials.

In Figs. 20, 22, and 26 dams with substantial rock toes are shown. In many locations, however, the rock with which to construct such big rock fills is not available except at prohibitive cost. Consequently, various schemes are utilized so that proper drainage may be obtained without going to the expense required to obtain a big rock toe fill.

If there is much hydrostatic pressure anticipated near the toe, it can be counterbalanced by moving the drain and filter farther upstream to get more weight on top of it, but in that case seepage will be increased owing to the shortening of the path of flow. If the drain is placed at the center line, there is a material increase in seepage, but this may be more than compensated for by the fact that the safety of the structure is greatly improved by better drainage.

Under many conditions the authors favor the use of a filter set back under the dam a distance of 30 to 50 per cent of the distance from the center line of the dam to the downstream toe. If the drain is properly constructed, there is ample safety against piping because of the great weight from the dam. Often the increase in seepage is not enough to be of economic importance.

Figs. 20 and 24 show drains which have been built back from the toe in the cross-section of the dam. If there is any danger of piping at the toe, the drains should be moved back to get greater weight on them.

27. Effect of Foundation Cutoff on Seepage. In many cases where an earth dam is proposed the foundation consists of alluvial deposits of pervious sands and gravels at and near the surface, with an impervious stratum (rock, clay, or silt) at a greater depth. Frequently the expense of a cutoff joining the impervious stratum in the foundation with the base of the impervious portion of the dam is not prohibitive. Figs. 1, 2, 17*b*, and 18*d* show sections of dams where a cutoff was constructed through a pervious foundation to an impervious stratum.

As an example, take Fig. 2 with minimum length of path without cutoff 400 ft, l. Permeability, coarse sand $1480 \times 10^{-4} = 0.1480$/ft per min, k. Depth to impervious layer 50 ft. Difference in head, h, headwater to tailwater = 85 ft. Permeability coefficient of clayey silt $0.06 \times 10^{-4} = 0.000006$ ft per min. Minimum length of path through cutoff = 20 ft.

Using Eq. 5 of Chapter 16, $q = k\frac{h}{l}A$, then with no cutoff,

$$q \text{ per foot of width} = \frac{0.1480 \times 85 \times 50}{400} = 1.57 \text{ cu ft per min}$$

and with cutoff of average thickness of 20 ft,

$$q \text{ per foot of width} = \frac{0.000006 \times 85 \times 50}{20} = 0.00128 \text{ cu ft per min}$$

In other words, with the cutoff the seepage through the foundation would be about one thousandth of what it would be without the cutoff. This reduction

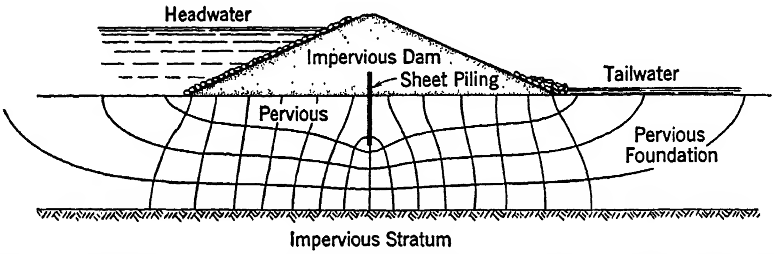


FIG. 29. Flow net for pervious foundation of impervious dam with steel sheet piling which does not completely penetrate pervious foundation.

in seepage may be of great importance in connection with the prevention of piping or it may be of economic importance because of the value of the water lost.

28. Steel Sheet Piling Cutoff. In many cases steel sheet piling is utilized for foundation cutoffs such as those described in Art. 27 for the reason that when the depth exceeds about 30 to 50 ft, it is practically always cheaper to drive steel sheet piling than to dig a trench and refill it with compacted impervious soil. At Kingsley Dam, Nebraska, steel sheet piling 125 ft long was driven to connect the core of the hydraulic fill dam to the impervious stratum below. (See Art. 42 and Fig. 38, Chapter 19.) With modern methods of driving and jetting it is feasible to obtain real assurance that the steel sheet piling reaches the recorded depth. By drilling holes each side of the piling it is possible to find out whether or not piling has curled or gone out at an angle. The kind of steel pile section chosen is important. If there are many boulders a straight heavy section is desirable, with a strong interlock such as U. S. M 108 weighing 35 lb per sq ft. At Kingsley Dam, however, where there were not many boulders, a section similar to M 112, which weighs only about 23 lb per sq ft, was used. It is not practicable to drive steel sheet piling so that it is absolutely tight. One should never expect it to be as tight as a cutoff trench filled with compacted impervious material. Thus in the example given in Art. 27, if steel sheet piling were used instead of a cutoff of impervious material, one could with careful workmanship and the use of interlock compound reduce the seepage to 1/5 or 1/10 of what it would be without any cutoff. With poor workmanship, or if the piles cannot be seated in the impervious stratum, the reduction might be only 1/2 or less. There

would be no chance at all of obtaining a reduction of seepage to $\frac{1}{1000}$ of the seepage without any cutoff as in Art. 27.

Jetting is generally essential in reaching material depths with steel sheet piling in sandy and gravelly material. In some cases in order to reach depths of 125 to 140 ft, it has been necessary to use jets discharging 200 to 300 gal per min at 300 ft head and in addition to use compressed air with the water. Two to three jets on each side of the piles being driven are sometimes used in a yoke.

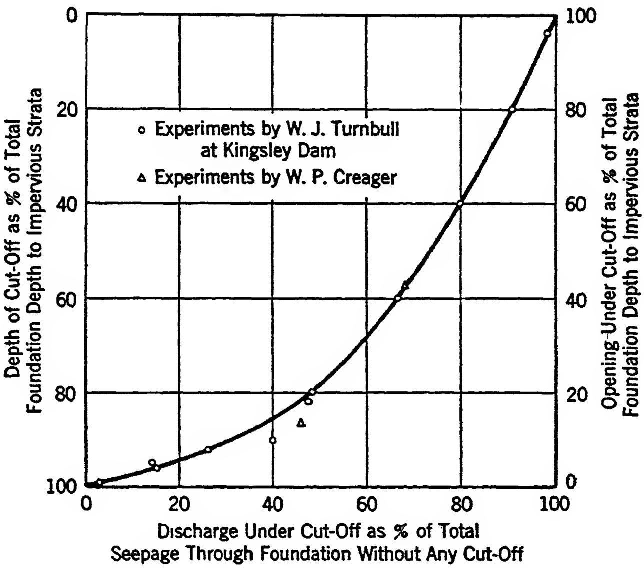


FIG. 30. Seepage under cutoff through foundation for various percentages of penetration to impervious strata. Seepage is stated as per cent of that which would occur without cutoff.

29. Partial Cutoff. A partial cutoff is one which extends down from the impervious section of a dam into the underlying strata, but does not reach an impervious stratum. In many cases it would be impracticable or extremely expensive to continue the cutoff to an impervious stratum, and, accordingly, the engineer should consider the use of a partial cutoff.

Owing to the fact that alluvial deposits are stratified and that, therefore, the horizontal permeability may be 10 to 50 times the vertical, the effect of such a partial cutoff in reducing seepage may be much greater than might at first appear probable. (See Art. 32, this chapter, and Art. 21, Chapter 16.)

Frequently in driving a steel sheet piling cutoff which is intended to reach an impervious layer, it is found impracticable to get all of the piles fully driven and seated in the impervious layers. This may be due to nests of boulders encountered or other causes. The seepage which will occur through such openings in homogeneous, isotropic materials is much greater than one would expect and is somewhat analogous to the flow through a partly closed valve.

The curve in Fig. 30 gives the discharge beneath the partly driven sheet piling or partial cutoff as a percentage of the discharge which would take place if there were no cutoff and the material was homogeneous. For instance, let us assume that the vertical distance from the base of the impervious section of the dam down to the impervious stratum under the dam is 100 ft and steel sheet piling is driven to a depth of 90 ft but cannot be driven further because of boulders. Then from Fig. 30, using 90 per cent as the ordinate, it is seen that the seepage which will take place will be 36 per cent of the seepage which would take place with no cutoff at all, assuming that the steel sheet piling or other cutoff is tight as far down as it goes. The total amount of seepage with no cutoff is determined by the same method as in Art. 27.

30. Upstream Blankets. Instead of using a cutoff (see Arts. 27, 28, 29, 31, and 32) under a dam on a pervious foundation, an impervious upstream blanket may sometimes be advisedly used. The purpose of such a blanket is to increase the length of the path of percolation for seepage under the dam and thus decrease the velocity and quantity of seepage.

In comparing the efficiency of a blanket with a partial cutoff, consideration must always be given to the fact that horizontal permeability is usually much greater than vertical permeability. (See Art. 32.)

In Fig. 31 is shown a dam with a pervious foundation and a relatively impervious natural or artificial upstream blanket. In the case of many dams, there is a natural blanket from which trees and other vegetation should be removed and then defective places patched and the entire surface rolled.

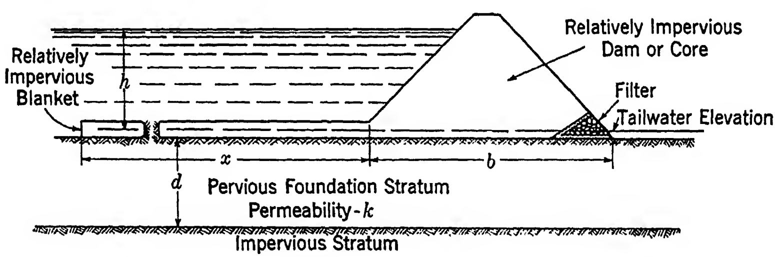


FIG. 31. Effect of impervious blanket.

The stream channel has frequently been completely eroded down to sand and gravel so that here a complete blanket must be constructed. Even a fully natural blanket should be thoroughly rolled with a sheepsfoot roller after removing the vegetation. The reason for this is that otherwise there may be numerous root holes or tubercles through the blanket to the pervious foundation. For usual conditions the upstream impervious blanket should be not less than 5 ft thick and there is seldom any good reason for requiring it to be over 10 ft thick.

The total seepage should not be greater than is economic for the given project. The path of percolation under the blanket and the impervious portion of the dam should never be less than 8 times the gross head and a ratio of 10 or more is

desirable.[19] The theoretical blowout or flotation gradient is 1 to 1, as discussed in Arts. 34 and 35. However, local piping may occur with much flatter over-all gradients. Properly graded and weighted filters are necessary to prevent escape gradients approaching 1 to 1 in all such cases, and the higher the seepage the more important are such details. (See Arts. 24 and 35.)

Usually the material available for the blanket is so tight in relation to the pervious underground that it is not necessary to consider flow through the blanket in determining the desired length of the upstream blanket.

In Fig. 31

$$x = \frac{khd - pQb}{pQ} \tag{17}$$

in which x = length of impervious upstream blanket in feet,
k = mean horizontal permeability coefficient of the pervious underground,
h = gross head in feet on impervious upstream blanket,
d = depth of pervious underground, in feet,
p = percentage (stated as a decimal) of flow under dam without a blanket to which it is desired to reduce the seepage by means of the blanket,
b = length of impervious portion of base of dam,
Q = flow under dam without a blanket per foot of dam = $k(h/b)d$

Example. Assume the following values: $k = 0.12$ ft per min (medium sand gravel), $h = 50$ ft, $d = 80$ ft, $p = 0.20$, $b = 200$ ft.

If there were no blanket at all the discharge under the dam would be $Q = \dfrac{0.12 \times 50 \times 80}{200} = 2.40$ cu ft per min per ft width of dam. Note that if the dam were 1000 ft long, the seepage under it would be 2400 cu ft per min (40 cu ft per sec) which we will assume it is economically desirable to reduce to about 20 per cent of this amount. Hence p is assumed as 0.20.

$$x = \frac{0.12 \times 50 \times 80 - 0.20 \times 2.4 \times 200}{0.20 \times 2.4} = 800 \text{ ft}$$

and

$$x + b = \frac{b}{p} = \frac{200}{0.20} = 1000$$

and the seepage under dam and blanket is $khd/(x + b) = 0.48$ cu ft per min per ft of dam. Thus, on the above basis the desired length of upstream blanket to be added is 800 ft. The ratio of path of percolation to head is $1000/50 = 20$, which more than meets the criterion.

If there is any doubt that seepage through the blanket might be enough to make it desirable to increase this length of blanket, it is a simple matter to com-

[19] See E. W. LANE, "Security from Underseepage Masonry Dams on Earth Foundations," *Trans. Am. Soc. Civil Engrs.*, Vol. 100, p. 1235.

pute approximately this seepage and if large enough to materially affect results, increase the thickness of the blanket or extend it further upstream. Thus, in the above example, assume that the relatively impervious blanket is 5 ft thick and is composed of medium silt which in place after rolling has a vertical permeability of 0.00002 ft per min. Assume that the average net head on the blanket is $h/2$ (actually it is somewhat less).

Then the discharge through the blanket by the Darcy equation is

$$q_1 = \frac{k_1 h(x + b)}{2t} \qquad [18]$$

in which q_1 is discharge through blanket, k_1 is vertical permeability coefficient of blanket, t is the thickness of the blanket, and the other factors are the same as in Eq. 17.

In the example $q_1 = \dfrac{0.00002 \times 50 \times 1000}{2 \times 5} = 0.10$ cu ft per min per ft of dam.

Thus the total seepage under dam and blanket is about $0.48 + 0.10 = 0.58$ cu ft per min. If desired this could be somewhat reduced by thickening the blanket, but evidently in this case we would get a much greater proportionate reduction in seepage by increasing the length of the blanket. However, the length of the blanket determined by Eq. 17 of 800 ft is already sufficient unless the water lost by seepage has an unusually high value. If the water lost by seepage does have a very high value consideration should be given to a total cutoff as discussed in Arts. 27 and 32.

31. Core Walls. Some years ago a large proportion of all earth dams constructed had core walls of masonry or concrete, and some engineers still feel that it is not entirely conservative to construct earth dams without a concrete core wall or cutoff. Regardless of whether the material in the foundation was pervious or impervious, it was common practice to excavate a trench to ledge rock and bond the core wall to ledge rock and carry it up with the compacted fill on each side of it to a point near the top of the dam. The earth dams of the Ashokan Reservoir, New York City Water Supply, utilize such core walls founded on ledge rock, the wall at one point being more than 100 ft below the original ground surface (see Fig. 32).

In the days before concrete became common it was usual to make a narrow core out of puddle, which was made by mixing damp clay very thoroughly with gravel. The mixture was then tamped thoroughly into place to form a puddle wall. These so-called puddle walls formed an excellent central impervious section for an earth dam, and the only reason for their discontinuance was the fact that it became cheaper to accomplish the same objective by other means. Yarrow Dam, Fig. 34, has such a puddle wall.

The use of concrete core walls was also an insurance against sloppy workmanship in construction. For instance, at a site the river bed might consist of stones and gravel on ledge rock. For an earth dam without a core wall, a sloppy contractor might just deposit the clayish or silty material which was to form the central portion of the dam right on top of the loose stones and boulders and thus

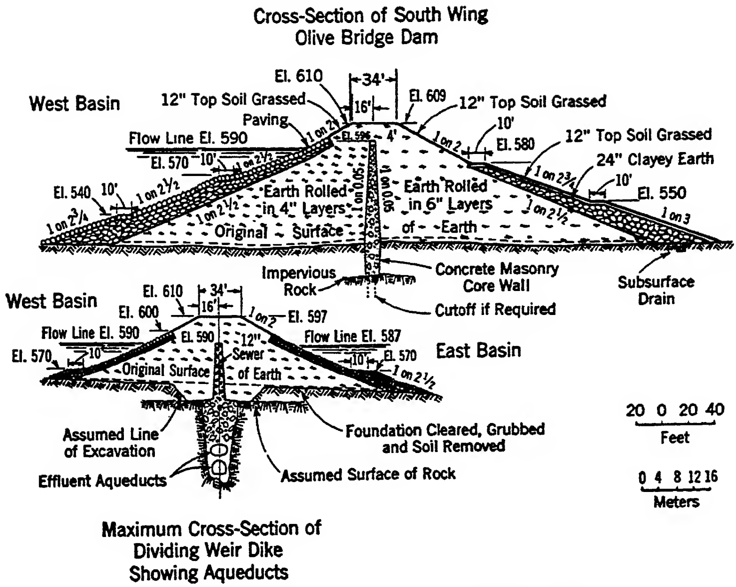


Fig. 32. Typical cross-section of Ashokan dikes.

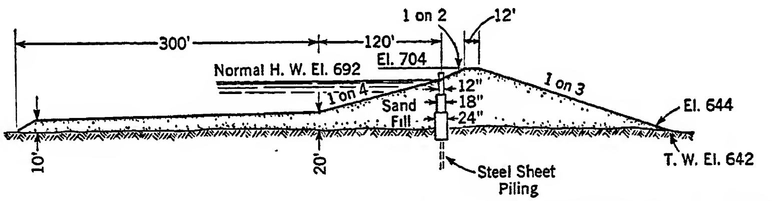


Fig. 33. Junction Dam showing reinforced concrete core wall on steel sheet piling cutoff

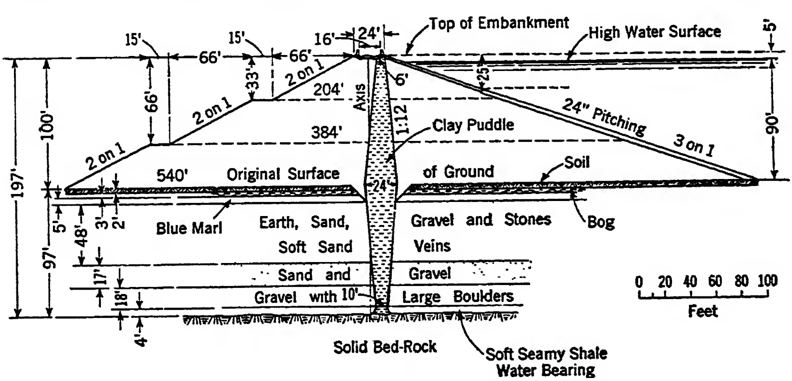


Fig. 34. Yarrow Dam (England); typical cross-section showing puddle core. (*From "Earth Dams," by Burr Bassell, Eng. News Publishing Co., N. Y., p. 10, 1907.*)

form a sort of French drain from the upstream face to the toe. Such a condition might very well result in serious piping. On the other hand, if a core wall were constructed at the center line and bonded to the ledge rock, there would be no danger of serious piping. However, the authors believe that the best insurance of all is competent engineering control of construction. With competent engineering control one could be sure that the loose rock and gravel would be removed and the impervious material of the central portion of the dam bonded to the ledge rock.

However, there are conditions under which it is advisable to use a concrete core wall; for instance, in a section of the country where there is no relatively

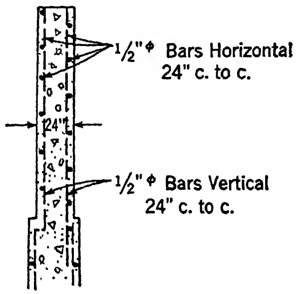


FIG. 35. Partial section of typical reinforced concrete core wall.

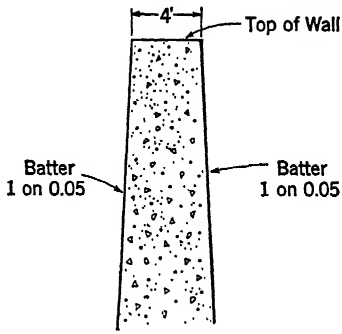


FIG. 36. Partial section of typical mass concrete core wall.

impervious material available except at prohibitive cost for transportation and where the only material available is relatively pervious sands and gravels, but where ledge rock is only 10 to 40 ft below the original ground surface.

A diaphragm type of reinforced concrete core wall can be constructed and founded on the ledge rock which will satisfactorily form the impervious central section of an earth dam, with the pervious sand and gravel thoroughly compacted to form the supporting shells on either side. Wissota Dam, Wisconsin, Fig. 19*a*, is such a dam, with diaphragm core wall of reinforced concrete founded on granite rock. Junction Dam, in Fig. 33, is another one of the same type except that here the thin reinforced concrete core wall is founded on a cutoff of steel sheet piling.

Fig. 35 shows a cross-section of a typical reinforced concrete core wall and Fig. 36 shows a section of a typical mass concrete core wall.

The thin reinforced concrete core walls should be constructed in lengths not over 30 ft long and in 10- or 12-ft lifts. Naturally this type of core wall cannot stand much unbalanced pressure. Hence the fill in each side should be brought up evenly and thoroughly compacted. Copper, aluminum, or rubber water stops should be used at vertical construction joints (see Fig. 37). Rubber is favored for locations that will always be moist and dark. Diaphragm reinforced concrete core walls of this type have been used successfully for many years under proper conditions.

Where there is good impervious material near the proposed dam site, its use for a central core will usually be found more economical and more desirable than the use of a concrete core.

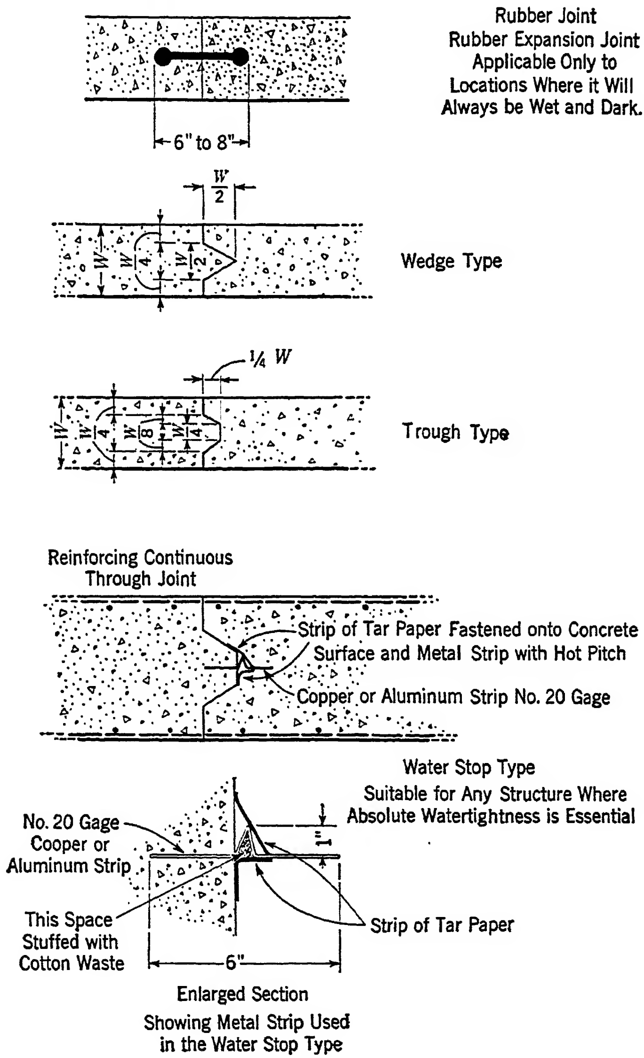


FIG. 37. Types of expansion joints used for core walls.

32. Cutoff Walls. Cutoff walls are usually differentiated from core walls in that cutoff walls are in or on the foundation and just reach into the impervious section enough to make a bond with it, whereas core walls generally extend up into the body of the dam a substantial distance. Cutoff walls, like core walls,

were much more used in previous years than they are today. As a matter of fact, if proper care is used in bonding the impervious section of an earth dam to

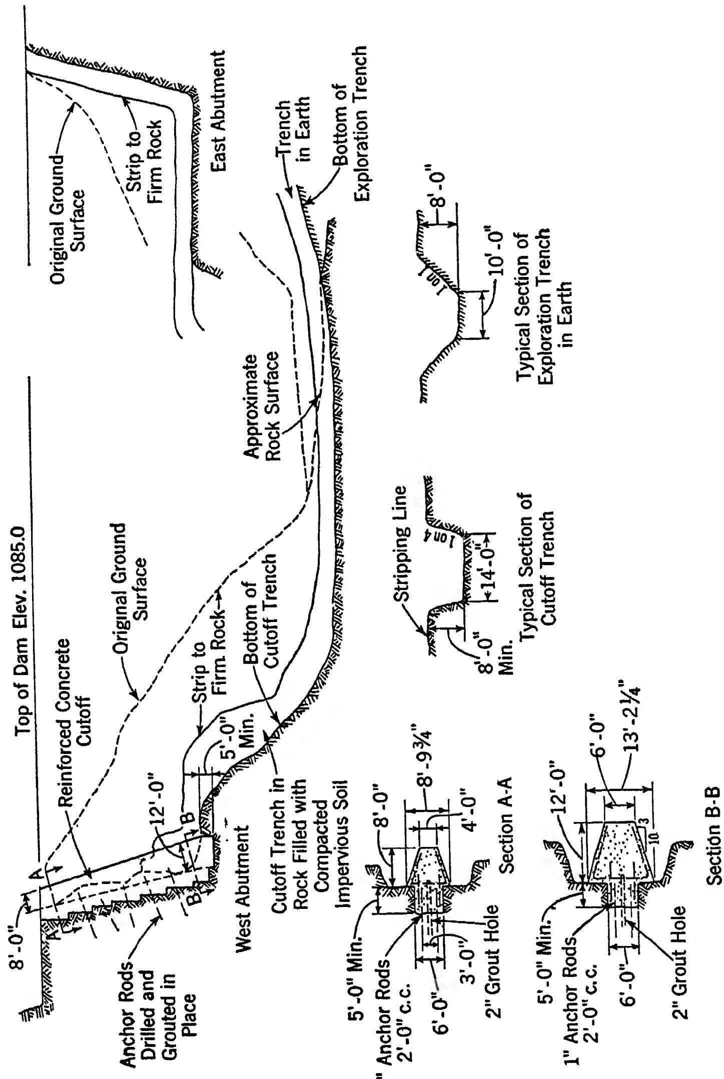


FIG. 38. Cutoff wall and cutoff trench, Pleasant Hill Dam, Ohio (1937).

the foundation of ledge rock, hardpan, clayish material, etc., more resistance to seepage can be obtained than with the use of the usual concrete cutoff, for the simple reason that clayish silt is tighter than concrete and also the length of contact is greater.

Cutoff walls are sometimes constructed from ledge rock or from any other relatively impervious stratum up through a pervious foundation stratum to make a bond with the relatively impervious section of the dam (see Fig. 38). As a rule, if the depth through the pervious foundation stratum to ledge rock or other impervious stratum is not more than about 30 ft, a trench refilled with relatively impervious soil, if available nearby, is usually the most economical way of obtaining a cutoff. If the depth materially exceeds 30 ft, it is usually more economical to drive steel sheet piling to form a cutoff. A cutoff constructed by trenching and refilling with compacted impervious material is shown in Fig. 2.

Cutoffs of steel sheet piling through a pervious foundation to an impervious stratum are shown in Figs. 13, 37, and 38 of Chapter 19.

Cutoffs which penetrate a pervious foundation only part way are not usually fully effective in reducing seepage, as shown in Arts. 28 and 29. However, such cutoffs have one extremely important advantage over an upstream blanket which enforces the same length of seepage path.

Practically all soils deposited by water are stratified or have lenses of material of various sizes. A vertical partial cutoff will penetrate a number of these lenses or strata and will thus force the seepage to pass through the different strata. Some of the strata may be much less pervious than the foundation material immediately under the dam. The net result of this is that reduction of seepage for the same length of seepage path may be from 4 to 20 times or more as great for the partial vertical cutoff as for the upstream horizontal blanket of impervious material.

When earth dams are constructed in a valley where the cross-section is entirely or largely ledge rock, cutoff walls of concrete or reinforced concrete bonded into the ledge rock in the bottom of the canyon or gorge and extending up the sides of the ledge rock are frequently used. These cutoff walls generally project 5 to 10 ft into the impervious portion of the embankment. In many cases such cutoff or baffle walls offset the effects of imperfect workmanship. In general, except where the top of the ledge is unusually firm and free from open seams, a trench cut into the ledge rock and refilled with compacted impervious soil is to be preferred. The sides of such trenches should have a slope not steeper than 6 vertical on 1 horizontal. However, this slope will not eliminate all overhangs, and the length of path of percolation should be considered to be only the contact of the fill with the bottom of the trench.

Where the trench runs up a steep abutment, as when the dam abuts against a nearly vertical cliff and a trench is excavated in the cliff to expose good rock, and in all other cases where the impervious earth element of the dam abuts solely against a nearly vertical rock cliff, any overhanging ledges of rock are practically certain to result in cavities between the rock and the earth. In such cases three expedients are possible:

a. Use cutoff walls as previously described.

b. Excavate the rock cliff on a slope so flat that no overhangs are possible. In some cases this would require a slope so flat as to be uneconomical.

c. Place a layer of concrete or gunite against the rock to obliterate overhangs.

33. Devices for Seepage Line Observation. For determining the elevation of the seepage line at various points in the cross-section of an earth dam the usual method adopted is to sink several vertical perforated pipes between headwater and the downstream toe of the dam. Sometimes a number of unperforated pipes are used with their bottoms terminating in pockets of sand. In cases where the hydrostatic pressure in a pervious stratum which lies below a relatively impervious stratum is desired, a well point is used at the bottom and the rest of the pipe is tight.

The piezometer pipes (or more properly seepage pipes) usually terminate at the ground surface on the downstream face of the dam or the top of the dam, where they are kept capped. Observations are preferably made by dropping a float down on a tape to the water surface and measuring the distance down from the known elevation at the top. Nicer methods have been devised, but great precision is not justified.

The perforated pipe and well points work perfectly in highly pervious materials, including sands and gravels. Care must be taken that openings into the pipe are smaller than the diameter of most of the particles in the surrounding material.

In using such pipe in coarse silts, an outside casing should generally also be used. The outside casing should also have holes throughout, and the space between the two pipes should be filled with sand so that a filter is formed which will prevent the silt from passing through the holes in the inside pipe or piezometer and clogging it.

The finer the material in the embankment the more difficult it becomes to obtain accurate observations on the position of the seepage line. Thus with some clays it may take a number of years after the completion of construction for headwater to seep through to the first piezometer pipe. Fig. 39 shows several types of piezometer pipes used for observing the position of the seepage line. The amount of water which seeps through the more impervious soils is so slight that it takes an extremely long time for the water level in the piezometer pipes to respond to changes in headwater elevation.

Pressure cells may be used for determining the approximate position of the seepage line. Because they require only an insignificant quantity of seepage water to actuate them, they are tremendously more sensitive than the usual piezometer pipes and they eliminate to a large extent the "lag." This makes them suitable for installation in embankment materials which are relatively tight.

The hydrostatic pressure cell developed at the U. S. Waterways Experiment Station is a recent type requiring little seepage water for operation. The cell is 3½ in. in diameter and 2 in. thick. A perforated plate with a screen soldered to it keeps the soil from coming in contact with the diaphragm but allows passage of the pore water to act freely on the diaphragm. The diaphragm is an integral part of a casing, the inside space of which contains two Baldwin Southwark S R-4 Bonded Metalectric strain gages. One gage, called the active gage, is cemented to the diaphragm and the other, called the dummy gage, is cemented to a metal

plate which is attached to the side wall so that it will not be subjected to any stresses from external pressure. The chamber formed by the casing and containing the gages is sealed from external soil, water, or air pressure by a top plate and a gasket, being fastened by screws down to the casing. The chamber has a gland attached to it, through which a three-wire electric cable passes, bringing the wires to a conveniently located terminal board. (See Fig. 40.) The theory of operation is as follows: The pore water pressure causes a deflection of the

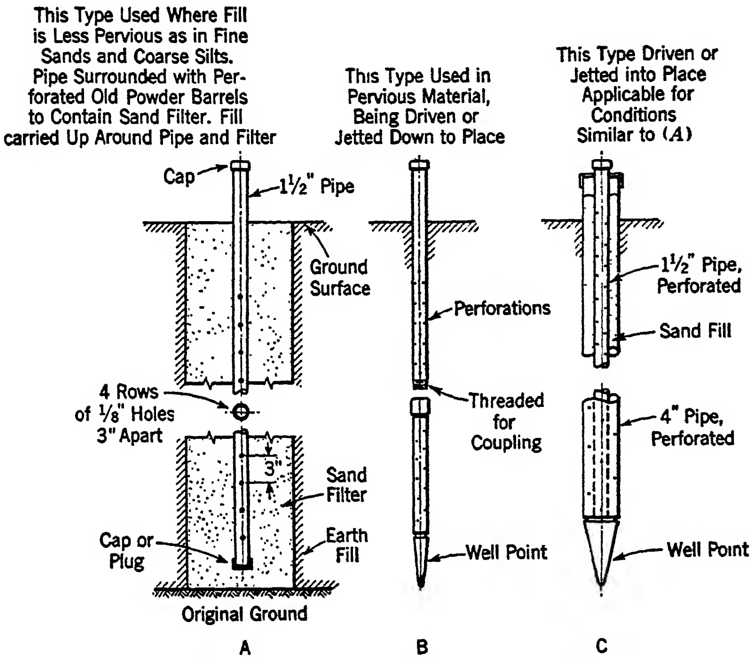


Fig. 39. Piezometer pipes for measuring position of seepage line.

diaphragm, the strain in which is measured by a change in resistance of the electric wire resistance strain gage. The active and dummy gages act as two arms of a Wheatstone Bridge and the recording bridge (described in Baldwin Southwark Bulletin No. 164) as the other two arms. If the recording bridge is initially balanced so that no current flows through the circuit, any unbalance due to a change in resistance of the active strain gage is reflected by an unbalance of the bridge, resulting in the deflection of the galvanometer. The necessary change in resistance in the recording box to rebalance the circuit is a measure of the deflection of the diaphragm and of the external pressure applied to the cell. The dummy gage also serves the purpose of compensating for temperature. Since it is identical with the active strain gage and is placed adjacent to it, any change in temperature that occurs will affect both gages in the same manner and will be balanced out in the electrical bridge circuit.

The above pressure cell has been thoroughly tested in the laboratory, and there are also several installations in service. Records obtained so far are excellent, but the cell has not been in service long enough to determine its probable life.

With this type of cell no flow of pore water to or from the cell is necessary to indicate the pressure. Also no change in the amount of pore water near the cell

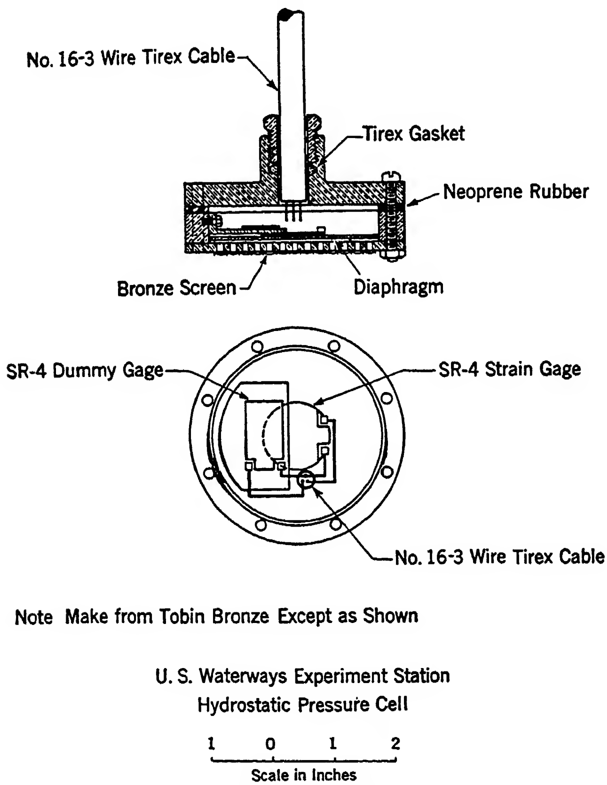


FIG. 40. Hydrostatic pressure cell, U. S. Waterways Experiment Station.

is required. Consequently this type of cell is particularly applicable for use where the material is relatively impervious.

As a means of locating the approximate seepage line, the cell is very useful except when it is located below an impervious stratum, in which case the hydrostatic pressure determined would give no inkling of the location of the seepage line.

34. Flotation Gradient and Quick Sand. Usually quick sand is a relatively uniform fine sand in a rather loose condition completely saturated and under some slight upward hydrostatic pressure. Actually, quick sand is a condition and not a material. Coarse sands and gravels are sometimes "quick." All that

is necessary is that the material should be completely saturated and have enough upward hydrostatic pressure on it to counterbalance the weight of the particles of the material.

On one dam with which the author was connected the foundation throughout most of the valley was a coarse gravel (in connection with which an upstream blanket was used). The gravel was under enough upward hydrostatic pressure that it was "quick" and one could sink down into it halfway to his knees by jiggling up and down a little bit. The question arose as to whether it would be possible to utilize the "quick gravel" as a foundation for the dam having a height of 115 ft. The embankment was constructed on it, leaving a vertical shaft through the embankment for the first 30 ft. Water rose in the shaft a few feet and after that loads placed on the "quick gravel" did not sink into it at all. In other words as soon as the upward relatively slight hydrostatic pressure was equalized this "quick" gravel was no longer "quick" but became a stable foundation with adequate shear resistance such as coarse gravel should have.

If water is passing upward through sand or gravel and the pressure and velocity of the water is gradually increased, a condition is reached where the pressure becomes enough to counterbalance the weight of the particles of sand and gravel in water. At this point there is a sudden loosening and swelling of the material. The sand or gravel is then in what is called a quick condition. It is in an unstable equilibrium, and a slight further increase in pressure will cause the material to flow away like water.

For this critical flotation gradient

$$i_F = \frac{h}{L} = (1 - P)(S - 1) \qquad [19]^{20}$$

in which i_F = flotation gradient,
h = difference in head,
L = length of path,
P = porosity or per cent voids expressed as a decimal,
S = specific gravity.

Using the above formula and assuming a specific gravity of 2.65, Harza determined critical flotation gradient for various porosities as follows:[21]

Porosity or Per Cent of Voids	Critical Flotation Gradient
0.30	1.15
0.35	1.07
0.40	0.99
0.45	0.91
0.50	0.825

[20] This equation from L. F. HARZA, "Uplift and Seepage Under Dams on Sand," *Trans Am. Soc. Civil Engrs.*, Vol. 100, 1935, p. 1358 (Eq. 6).

[21] Idem, p. 1376.

35. Piping. If the head increases slightly over that which gives the flotation gradient of Eq. 19 the material will start flowing, and piping is in progress.

Piping occurs when seepage water issues from an embankment or ground surface under sufficient pressure and with sufficient velocity so that the particles comprising the material are carried away.

A number of piping experiments have been made to determine the escape gradient or hydraulic gradient near the point of egress at which piping may start. All the experiments with which the authors are familiar, in general, substantiate Eq. 19. Among these experiments are those of Terzaghi,[22] Philippe,[23] and Middlebrooks.[24]

In some of Philippe's experiments boils began to show up when the escape gradient reached 0.6 to 0.8. Boils began to form, but actual piping with a final blowout failure did not occur until the escape gradient h/l exceeded 1.5. There was one exception to this: where the material was a fine sand, complete failure occurred at a gradient of 1.04.

Vertical piping sometimes occurs at the downstream toe of an earth dam or levee. Usually in such cases there is a layer of material in the foundation which

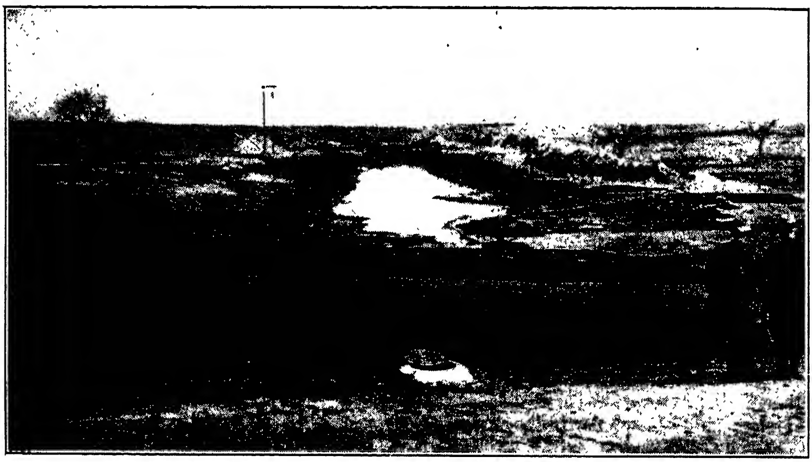


FIG. 41. Vertical piping at toe of an earth dam. Note boils on far side of pool.

is much more pervious than the strata at the surface of the foundation, and as the head increases the hydrostatic pressure in this layer increases until the water lifts the overlying strata, forming boils on the surface of the ground.

Boils in themselves are not necessarily disastrous. The authors know of one dam which had boils at its downstream toe for several years without disastrous

[22] *Am. Inst. Mining Met. Engrs.*, Tech. Pub. 213.

[23] R. R. Philippe at Zanesville Laboratory of U. S. Army Engineers (unpublished).

[24] T. A. Middlebrooks at Fort Peck Laboratory of U. S. Army Engineers.

results. In each one of the inverted cones which forms a boil the material is in a condition of flotation or suspension; individual particles rise up and fall back. The head increases a bit and then the particles in the cone rise higher and flow over the edge of the cone or boil and flow away. This is then a condition of incipient vertical piping.

With this condition, to save the dam it is necessary to quickly ring the boils with sand bags, put in a low auxiliary dam, or adopt other means so that some back pressure will be exerted on the boils. If sufficiently prompt action is taken it is usually possible to establish a condition of stability, so that the boils will just go on boiling and the water will flow out of the boils, but it will not carry material with it. In Fig. 41 is shown a case of incipient vertical piping.

To make a permanent repair, coarse sand and gravel is first placed over the area in which the boils occur and then covered with coarser stone until the added weight overcomes any tendency toward movement even at maximum head. Local vertical piping may occur even if the path of percolation is very long in relation to the head because all that is necessary is that the escape gradient should be very steep. Such local piping remote from the dam is generally not serious.

Horizontal piping occurs where the seepage water issues from the downstream face of an earth dam or natural bank with enough force and velocity to carry away particles of the materials of which the dam or the bank is composed. With horizontal piping the water sometimes almost gushes from the face and erodes the downstream face below the point of egress. The horizontal piping soon results in the formation of a small tunnel running back into the dam with the roof continually falling in and being carried away by the piping water.

Once horizontal piping has started the only remedy is to dump rock grading from fine to coarse right into the downstream face where the horizontal piping is occurring, so that an improvised drain and filter will be formed and the piping stopped. Both horizontal and vertical piping are serious and may lead to complete failure of the dam if not promptly corrected.

The possibility of serious piping may be prevented by having the path of percolation sufficiently long in relation to the head, thus reducing the hydraulic gradient, and by providing properly designed and constructed filters and drains so that a dangerous escape gradient will be avoided. (See also Arts. 21, 24 and 30.)

For safety against piping in dams the minimum ratio of length of path to head should be not less than $l/h = 5$, which, according to Eq. 19, with specific gravity $= 2.65$ and per cent of voids $= 50$ per cent would mean a theoretical factor of safety of 6.

In case of highly pervious foundations without cutoffs the minimum ratio of l/h should be not less than 8 for the foundation and 10 is preferable in many cases. (See Art. 30.)

Under special conditions where drainage and filter systems are specially designed for the given conditions lower ratios of l/h may be acceptable.

36. Cause of Free Passage of Water Through Dams. Seams or openings through an earth dam permitting the free passage of water may be caused in a number of ways, chief among which are the following:

1. By water following the exterior surfaces of pipes or conduits through the embankment.

2. By burrowing animals, such as muskrats.

3. By the placing of very pervious material containing large stones in an otherwise impervious embankment in such a manner as to make a blind drain from the upstream to the downstream face.

4. By failure to bond and compact the succeeding layers of the embankment properly.

5. By failure to bond the lower layers of an earth dam properly to the foundation.

6. By water following the smooth surfaces of concrete abutments or other concrete structures or passing through "French drains" accidentally created along the sides of such structures by deposition of gravel and stones.

37. Pipes or Conduits Passing Through Earth Dams. Pressure pipes or conduits passing through earth dams have been a source of trouble. Water from the reservoir has a tendency to follow along the smooth surface of such pipes or conduits. Even if the pipe through the embankment is well provided with concrete cutoff walls, the embankment may settle, and, if the settlement is unequal, the pipe will be disturbed and distorted, the joints may leak or even break apart, possibly causing a blowout and the destruction of the embankment.

All pressure conduits through earth dams require special consideration in design and construction. It is often desirable to require pressure conduits in an earth dam to be placed inside a concrete conduit or tunnel with free space around them.

The questionable procedure of supporting the pipe on piers from the foundation is sometimes adopted. If the pipe between supports is not sufficiently strong to support the embankment above, it may break when the material settles from under it. If the pipe between supports is sufficiently strong to sustain the load, the earth shrinking away from the under side leaves an opening for the free passage of water.

The author strongly believes that pipes and conduits should never pass through an earth embankment unless they rest in the original foundation material or are built on a battered solid masonry wall as thick as the outside diameter of the conduit (provided with cutoffs) founded on the original foundation material and extending throughout the embankment. The best practice is to place pipes and conduits in trenches excavated in the original foundation material or in a trench or tunnel through rock excavated through the hill at the side of the dam. Where pipes are used they should be embedded in concrete placed in the pipe trench at least up to the horizontal diameter of the pipe, in addition to being provided with suitable collars in the manner indicated in Fig. 42.

Pipes placed in trenches excavated in the foundation should have several concrete collars. Where the concrete collars are to be constructed a cross trench

should be dug in the bottom and into the sides of the main trench at least 24 in. Exposed faces of collars should have a vertical batter not steeper than 1 on 10 so that the embankment, when deposited, will tend to reach tighter and tighter contact with it as consolidation progresses. Forms should be built so as to block only the main trench, thus allowing the concrete to fill all the space of the cross trench. The concrete collar should be built at least 2 ft above the top of the main trench. The concrete used for collars should be of about 1 : 2 : 4 mix or equivalent and just wet enough to tamp readily, as more water will cause shrinkage.

There should be not less than three cutoffs throughout the width of the impervious portion of the embankment. After the cutoff forms have been removed,

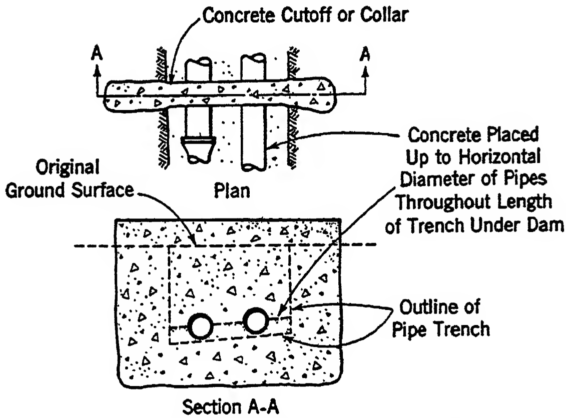


FIG. 42. A suitable method of protecting pipes passing under dam.

the pipe trench should be carefully refilled; the earth being deposited in layers 3 or 4 in. thick and rammed thoroughly, with compressed air power tampers, before the next layer is placed (see Fig. 43). Before the next layer is deposited, the material in place should be lightly sprinkled with water, if it is not sufficiently moist, so as to insure a bond between layers. Trenches of this kind should not be puddled, as puddling may cause the fill to shrink away from the walls of the trench.

Puddling, as generally practiced, is a very different process from that followed in constructing the puddle walls or puddle cores of many of the older earth dams. Usually an excavation is filled nearly full of water and then the earth simply dumped in. This procedure is seldom permissible and in many cases is positively dangerous.

It would seem that the construction of a concrete collar is so simple as to require little attention. However, the author has seen many defective collars constructed. In some cases, the cutoff did not extend to the walls of the trench, whereas in others the concrete merely touched the sides of the trench instead of projecting into them. The partial failure of the Table Rock Cove Dam in

South Carolina (see Table 1) forcibly illustrates the importance of providing suitable protection for pipe lines passing through an earth dam.

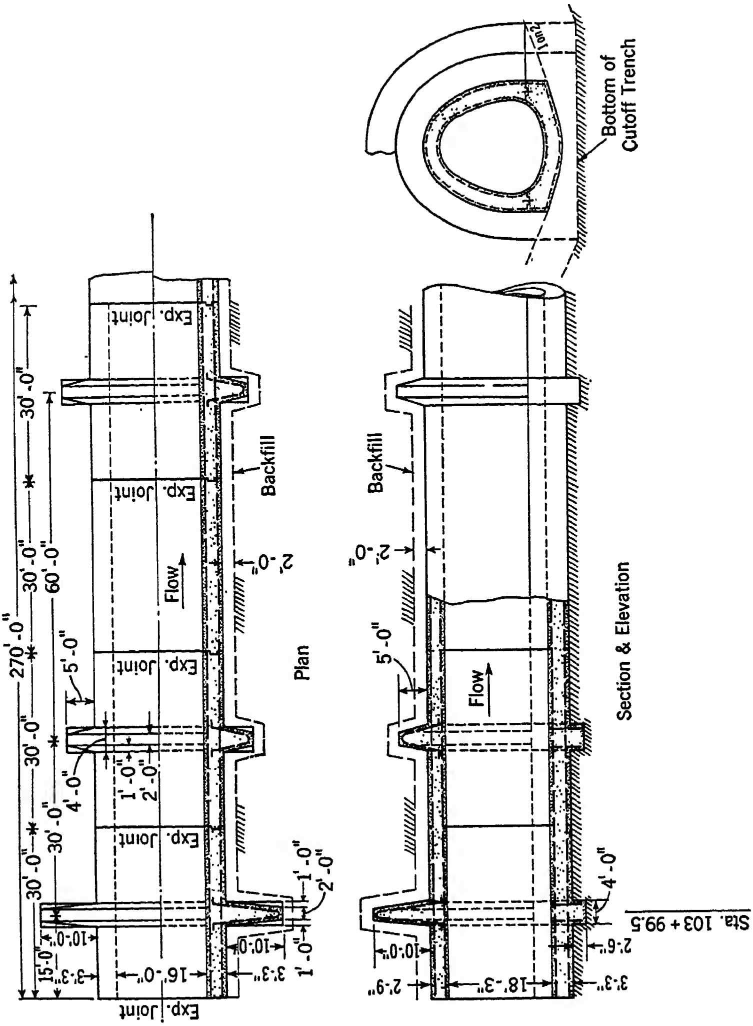


FIG. 43. Suitable protection against seepage along conduit. Cutoff collars at Sardis Dam. Refill around collars and conduit tamped in place by air tampers.

Control of outlet pipes or conduits should always be from the reservoir side by means of suitable valves or gates. With this arrangement it is possible to repair defects which may develop in a conduit.

38. Protection Against Burrowing Animals. Burrowing animals, particularly muskrats, have been blamed for the failure of many earth dams. Woodchucks, snakes, beavers, eels, crawfish, mice, and rats are also said to have injured earth dams. The muskrat, however, apparently causes more damage than any of the

others. Many of the recorded early failures in America are said to be due to his burrowing habits. In the days of the old canals, he was a source of trouble to those charged with the maintenance of the embankments. Whenever a mill dam failed and the reason was not entirely clear, the muskrat received the blame. It is probable that many earth dam failures attributed to burrowing animals were really due to some other cause. Most failures attributable to burrowing animals have occurred in small and unimportant structures.

With the much greater top widths of earth dams and the greater freeboard, the possibility of serious injury from any burrowing animal is slight. The authors know of no authentic case where the failure of a large modern earth dam was due to burrowing animals.

39. Accidental Blind Drain Through Embankment. The careless placing of materials occasionally causes what is, in effect, a blind drain from the upstream to the downstream face. Such a condition is avoided by preventing coarse material—stones and coarse gravel—from forming a continuous line from the upstream to the downstream face. There is often some chance of something of this sort happening along the walls of a canyon or along a conduit.

40. Passage of Water Through Uncompacted Material. If succeeding layers of the embankment are not properly compacted and bonded there is a chance for water to pass through the loose material. After work on an embankment has been suspended for a time, the surface may become hard and smooth. If additional layers are placed without special precautions to secure a bond between the new work and the old, there will be danger of a leak along the plane of contact with the pervious uncompacted layer.

41. Bonding Earth Dams to Their Foundations. It is of the utmost importance to bond the lower layers of earth dams properly to the foundation material in order to prevent the passage of water along any possible dividing plane between the foundation and the dam. All perishable material, such as stumps, brush, sod and large roots, should be removed from the foundation. All top soil containing more than 10 per cent of vegetable matter should also be removed from the base of the dam.

Earth dams are sometimes built without top soil or even the sod being removed. Such practice is reprehensible for reasons previously stated. Some engineers are so particular about the removal of all vegetable substances that they require the removal of all roots, no matter how small or how widely separated. As a matter of fact, small segregated roots in an embankment or foundation could not do the slightest harm.

42. Cutoff Buttresses on Spillway and Powerhouse Walls. Spillway abutments, powerhouse walls, or other concrete walls extending through the dam in an upstream and downstream direction should be provided with cutoff buttresses projecting well into the embankment. There should be at least two buttresses on each wall passing through the embankment. It is unnecessary to make them more than 12 in. thick if they serve only as cutoffs, but they should be lightly reinforced in order to prevent separation from the main wall. The cutoff walls and the rear face of the main wall should have a slight batter (about 1 on 10), so

that shrinkage of the embankment will tend to bring the earth in closer contact with the concrete. This is an important detail since, with vertical walls, there may be a distinct tendency for the earth to shrink away from the concrete.

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CHAPTER 18

STABILITY OF EARTH DAMS

1. General. The stability of a dam is that property which enables it to stay in position. A dam is stable if the resultant of all the forces acting on the dam does not result in movement. Thus there are degrees of stability. If the forces resisting movement are in exact balance with those forces tending to produce movement, the dam would be barely stable and the factor of safety would be unity. This would be a dangerous condition because the slightest increase in the acting forces would result in failure. On the other hand, if the ratio of the forces resisting movement to those tending to produce movement is 1.5, we have a factor of safety of 1.5, which is generally considered adequate in structures of earth or rock.

In comparison with the usual determined factors of safety for structures of concrete and steel, the factors of safety of 1.3 to 1.5 generally considered acceptable in the design of earth dams seem very low. There are several reasons for the apparent lowness of the acceptable factor of safety in earth dam designs, some of which may be mentioned:

a. The figures used for the strength of earth materials are necessarily taken quite close to the minimum values obtained, and the strength which would be mobilized before disastrous failure could occur would usually be much greater.

b. Usually the factor of safety increases with the passage of time owing to consolidation, etc., so that a factor of safety which was originally 1.3 may eventually become 2.

c. In most cases the forces tending to produce movement are taken at the upper possible limit but may actually be materially less than the assumed values.

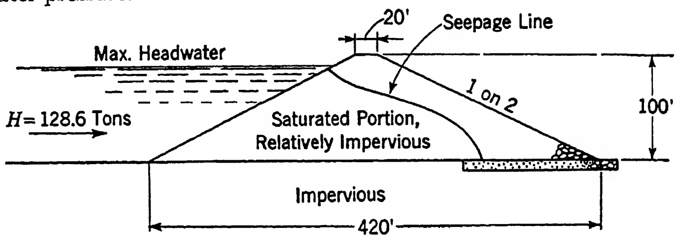
The net result is that if the investigation and analysis have been thorough, we usually have rather conclusive evidence that the factor of safety determined is the minimum to be expected.

Chapter 17 was devoted very largely to the effects of seepage through and under earth dams and to methods of minimizing and controlling seepage. Hence, in the present chapter it will not be necessary to devote much time to the effect of seepage on stability. The present chapter will provide methods of satisfying the following criteria of design 3, 4, and 5 as listed in Art. 7, Chapter 17.

In this chapter the authors first present rough and approximate methods for determining the stability of earth dams because the concepts of Coulomb and Rankine involved in such methods are already familiar to practically all engineers. Because these methods deal with plane surfaces only and there is generally a curved surface along which resistance to shear may be less, the required

factor of safety when using such methods should, in general, not be less than 2. Such methods are suitable for use in preliminary investigations, but the results obtained by them should be checked by the more precise methods discussed later herein for the final analyses of stability.

2. Stability of Earth Dam Against Headwater Pressure. The principal concern of the designer of masonry dams is to make the dams entirely safe against headwater pressure, whereas with earth dams we seldom have to worry about headwater pressure. The reason for this is that if an earth dam meets the criteria of design given in Art. 7, Chapter 17, it will be found to be safe against headwater pressure.



Wt. of Material in Dam

Dry Wt.	118 lb/cu ft
Saturated Wt.	137 lb/cu ft
Submerged Wt.	74.5 lb/cu ft
Moist Wt.	122 lb/cu ft

FIG. 1. Effect of headwater thrust on stability (see Art. 2).

It is believed that the example illustrated in Fig. 1 will make this clear. The dam is 100 ft high, the slopes are 1 on 2, and the maximum headwater is at an elevation of 90 ft above the base. The position of the seepage line results in 65 per cent of the cross-section being saturated and submerged.

From the data given in Fig. 1 the average unit weight may be computed thus:

65% of material submerged 0.65×74.5	= 48.5 lb
35% of material moist 0.35×122	= 42.6 lb
Average unit effective weight of cross-section	91.1 lb/cu ft

The area of the cross-section is $\frac{20 + 420}{2} \times 100 = 22{,}000$ sq ft. The effective weight of a section of the dam 1 ft wide will be

$$\frac{22{,}000 \times 91.1}{2000} = 1002.1 \text{ tons, an average pressure of } \frac{1002.1}{420} = 2.40 \text{ tons per sq ft}$$

We will assume that tan ϕ of the material of which the dam is composed is 0.3 and neglecting cohesion, the shear resistance is 1002.1 tons $\times$ 0.3 = 300.6 tons (Note: if tan ϕ were much less than 0.3, slopes would be much flatter.)

The headwater pressure will be perpendicular to the upstream slope, but the horizontal component of this pressure will be

$$H = \frac{wh^2}{2} = \frac{62.5 \times 90 \times 90}{2} = 251{,}000 \text{ lb} = 125.5 \text{ tons}$$

Thus the over-all factor of safety against horizontal shear due to headwater pressure is $\frac{300.6}{125.5} = 2.41$. The average shear due to headwater pressure $= \frac{125.5}{420} = 0.297$ ton per sq ft.

Usually the headwater pressure in an earth dam is not taken at the upstream face in accordance with the usual concept for masonry dams but is dissipated in the form of friction throughout the path of flow through the structure.

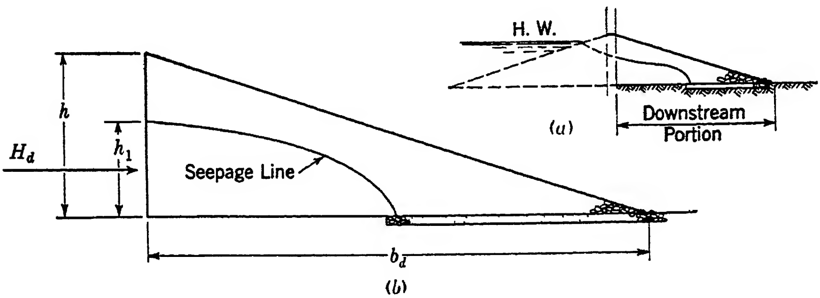


FIG. 2. Shear in downstream portion of dam (see Art. 3).

3. Horizontal Shear in Downstream Portion of Dam. The following is a simple method of determining the approximate horizontal shear stress in an earth dam at any given elevation.

In Fig. 2 a vertical plane is passed through the cross-section of an earth dam; we will consider for the present only the downstream portion of that cross-section.

Let w = the mean unit weight of the embankment material in the central section of the dam. In connection with determining the value of w, take the height of material which is below the seepage line at its submerged (or buoyant) unit weight and the height of material above the seepage line at the center of the dam at its moist or dry unit weight, and then take the weighted mean as w.

w_1 = the equivalent liquid weight per cubic foot of the material of which the dam is composed $= w \tan^2 (45° - \frac{1}{2}\phi)$.

w_w = unit weight of water, 62.5 lb per cu ft.

h = vertical distance from top of dam down to base or to the horizontal plane under consideration.

h_1 = vertical distance from seepage line down to base of dam on horizontal plane under consideration.

b_d = horizontal distance from top to bottom of downstream face.

S_d = average unit shear on downstream half of base of dam on horizontal plane through dam per 1 ft width of dam.

S_{md} = maximum unit shear on downstream half of base of dam or on a horizontal plane through dam corresponding to h.

ϕ = angle of internal friction of the material in the dam.

H_d = total horizontal shear on the downstream portion of dam in accordance with Rankine.

$$\frac{w_w h_1^2}{2} = \text{pressure of headwater}$$

$$H_d = \frac{w_1 h^2}{2} + \frac{w_w h_1^2}{2}$$

$$H_d = \frac{h^2 w \tan^2 (45° - \tfrac{1}{2}\phi)}{2} + \frac{w_w h_1^2}{2} \qquad [1]$$

$$S_d = \frac{h^2 w \tan^2 (45° - \tfrac{1}{2}\phi)}{2b_d} + \frac{w_w h_1^2}{2b_d} \qquad [2]$$

As indicated in Art. 8 the maximum unit shear may be twice the average; hence

$$S_{md} = \frac{h^2 w \tan^2 (45° - \tfrac{1}{2}\phi)}{b_d} + \frac{w_w h_1^2}{b_d} \qquad [3]$$

The above Eqs. 1, 2, and 3 are directly applicable to the downstream half of an earth dam only.

4. Horizontal Shear in Upstream Portion of Dam. The most severe condition to which the upstream portion of the dam can be subjected is to have the water

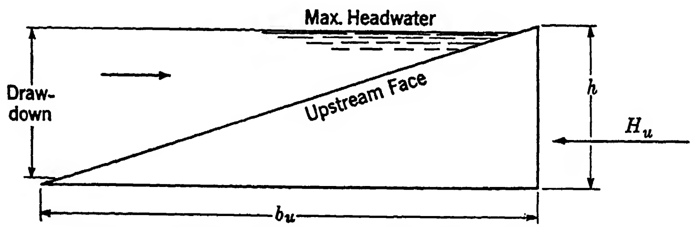


FIG. 3. Shear in upstream portion of dam with sudden drawdown (see Art. 4).

suddenly and instantaneously drawn out of the reservoir. Inasmuch as to have this happen would require the failure of some of the parts of the structure, this criterion is sometimes objected to as far too severe.

However, with very impervious materials the internal pressures that would result in the upstream portion of an earth dam from drawing down the reservoir, say, 10 ft in a week would not be greatly different from what it would be if the reservoir was drawn down 10 ft instantaneously.

If the material in the upstream part of the dam, on the other hand, is relatively clean rock or gravel, it may be assumed that it will drain just as quickly as the reservoir can be drawn down.

In general in case of sudden drawdown (see Fig. 3), the upstream portion of the dam would still be saturated on its completion, and, accordingly, one should utilize the saturated weight of the material for all material below the maximum seepage line when computing the shear. On the other hand, when it comes to computing the forces resisting shear, the unit weight utilized for all materials below the maximum seepage line will be the submerged (or buoyant) unit weight of the material except that clean rock and clean coarse gravel may be taken at its dry or moist unit weight both in computing shearing forces and in computing resisting forces. (See also Art. 20, this chapter.)

w_b = submerged unit weight of material in upstream portion of dam.
w_s = saturated unit weight of material in upstream part of dam.
b_u = horizontal distance from top to bottom of upstream face.
H_u = total horizontal shear on upstream portion of dam.
S_u = average unit shear over upstream part of dam.
S_{mu} = maximum unit shear over upstream part of dam.[1]
For h, h_1 and w_w see section 3

$$H_u = \frac{h^2 w_s \tan^2 (45° - \frac{1}{2}\phi)}{2} + \frac{w_w h_1^2}{2} \qquad [4]$$

$$S_u = \frac{h^2 w_s \tan^2 (45° - \frac{1}{2}\phi)}{2b_u} + \frac{w_w h_1^2}{2b_u} \qquad [5]$$

$$\text{maximum unit shear } S_{mu} = 2S_u = \frac{h^2 w_s \tan^2 (45° - \frac{1}{2}\phi)}{b_u} + \frac{w_w h_1^2}{b_u} \qquad [6]$$

The position of the maximum unit shear stress may, without serious error, be taken at a point 40 per cent of the horizontal distance from the top of slope to the point where the slope intersects the horizontal plane or base under consideration.

5. Factor of Safety of Earth Dam Against Horizontal Shear. The resisting force against the horizontal shear on the downstream portion of the dam, Fig. 2 is

$$R_d = W_{ed} \times \tan \phi + cb_d \qquad [7]$$

in which R_d = total force resisting shear in downstream portion of dam, W_{ed} = total effective weight of downstream portion of dam, c = cohesion or no load shear resistance per unit of area, and the other symbols have the same meaning as in Arts. 3 and 4. In computing W_{ed}, the unit weight of the material below the seepage line should be taken at submerged unit weight (w_b) and that above or outside of the seepage line at dry unit weight. Then

$$F_d = \frac{R_d}{H_d} \qquad [8]$$

in which F_d is the average factor of safety against shear in the downstream portion of the dam and H_d is total horizontal shearing force on the downstream portion of the dam as in Art. 3. Because the horizontal plane is not usually the

[1] Symbols whose meanings are not here designated have the same meaning as in Eqs. 1. 2, and 3 of this chapter.

weakest plane for failure, Eq. 8 should give a factor of safety of at least 2 in order for the design to be satisfactory.[2]

For the upstream portion of the dam the assumption of sudden drawdown will be utilized as the most severe (see Fig. 3).

$$R_u = W_{eu} \times \tan \phi + cb_u \qquad [9]$$

in which R_u = total force resisting shear in upstream portion of dam, W_{eu} = total effective weight of upstream portion of dam, and the other symbols have the same meaning as heretofore.

In computing W_{eu}, the unit weight of the material in the upstream portion of the dam below the seepage line should be taken as the submerged (or buoyant)

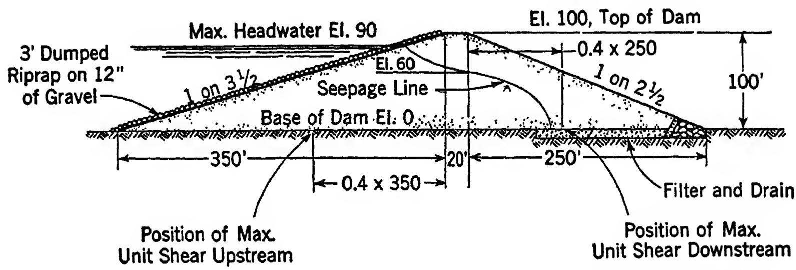


FIG. 4. Assumed design for determining safety factors in horizontal shear (see Arts. 6 and 7).

unit weight except that stone and coarse clean gravel may be taken at unit dry or moist weight. Material always above the seepage line may be taken at dry or moist unit weight. Then

$$F_u = \frac{R_u}{H_u} \qquad [10]$$

In Eq. 10 F_u is the average factor of safety against horizontal shear in the upstream portion of the dam and the other symbols have the same meaning as heretofore.

F_u is also referred to as the factor of safety against sudden drawdown. Because there is generally some other plane which is somewhat weaker than the horizontal, the factor of safety shown by Eq. 10 should be at least 2[3] for the design to be considered satisfactory.

6. Safety Against Downstream Horizontal Shear. *Example.* As usual we will just assume a design and then analyze it to see if it is satisfactory in the stated respect. Accordingly, we take the dam whose cross-section is given in Fig. 4. The case is relatively simple as the dam is of homogeneous material consisting of a medium silt placed in 6-in. rolled layers. The upstream slope is 1

[2] Utilizing the dangerous circle method, a factor of safety of 1.5 is generally considered satisfactory.

[3] When the dangerous circle method is used a factor of safety of 1.5 is generally considered satisfactory.

on 3½ and the downstream slope is 1 on 2½. The dam is 100 ft high and is founded on a firm and relatively impervious foundation.

ASSUMPTIONS:

Void ratio, material in dam, $e = 0.43$

or per cent voids (or porosity) $n = \dfrac{e}{1 + e} = 0.30$

spg (specific gravity) = 2.65

Hence unit weight per cubic foot (dry) =

$62.5 \times 2.65 \times (1 - 0.30) =$	116 lb/cu ft
Water contained in pores when saturated $0.30 \times 62.5 =$	18.8
Saturated unit weight	134.8
Deduct weight of same volume of water	62.5 lb/cu ft
Submerged or buoyant weight per cubic foot	72.3 lb/cu ft

Moist weight per cubic foot will be assumed as 120 lb/cu ft

Shear tests have been made on the material at the given density or weight per cubic foot, and it has been found that the value of c (unit cohesion) is so slight that it may be neglected and that the angle of internal friction is 26°. Thus $c = 0$.

$$\phi = 26^\circ \quad \tan \phi = 0.488$$

We will first find mean w for Eq. 1. Up to elevation 60 the material is below seepage line [4] and will have submerged (or buoyant) unit weight of 72.3 lb/cu ft and above and beyond the seepage line unit weight will be taken as moist or 120 lb/cu ft.

$$w = \frac{40 \times 120 + 60 \times 72.3}{100} = 91.50 \text{ lb/cu ft, using Eq. 1}$$

$$H_d = \frac{10{,}000 \times 91.50 \tan^2 (45^\circ - 13^\circ)}{2} + \frac{w_w h_1^2}{2}$$

$$45^\circ - \tfrac{1}{2}\phi = 45^\circ - 13^\circ = 32^\circ$$

$$\tan 32^\circ = 0.625 \text{ and } \tan^2 32^\circ = 0.391$$

$$\frac{w_w h_1^2}{2} = \frac{62.5 \times 60 \times 60}{2} = 112{,}600 \text{ lb}$$

$$H_d = 5000 \times 91.50 \times 0.391 + 112{,}600 = 291{,}600 \text{ lb} = 145.8 \text{ tons}$$

which is the total horizontal shear per foot of width tending to move the downstream section of the dam downstream. Fig. 5 may be conveniently used for obtaining $w_1 = w \tan^2 (45^\circ - \tfrac{1}{2}\phi)$.

Next we must determine the resisting forces downstream. Total area of downstream portion of dam is $\dfrac{100 \times 250}{2} = 12{,}500$ sq ft and under the seepage

[4] For the determination of the seepage line see Arts. 10 to 17, Chapter 17.

line [5] = 60 × 100 × 0.67 = 4030 sq ft of submerged material, leaving 8470 sq ft of moist material. Thus per foot of width there will be

$$
\begin{aligned}
8470 \times 120 \text{ lb} &= 1{,}017{,}000 \text{ lb} \\
4030 \times 72.3 \text{ lb} &= \underline{\quad 291{,}600} \\
&\quad 1{,}308{,}600 \text{ lb} = 654.3 \text{ tons}
\end{aligned}
$$

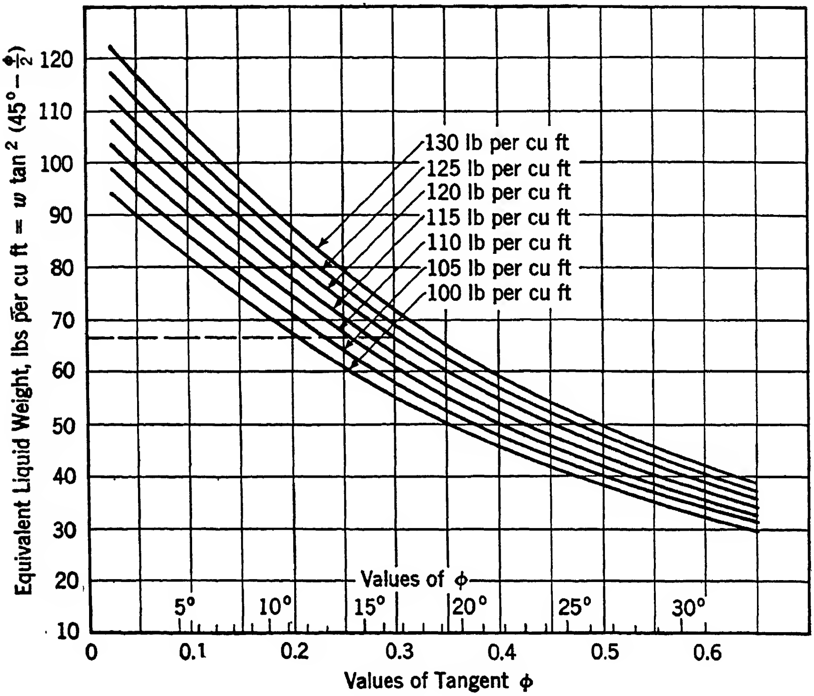


FIG. 5. Curves of equivalent liquid weight for various angles of internal friction (ϕ). (From Rankine's formula, equivalent liquid weight = $w \tan^2 (45° - \frac{1}{2}\phi)$ where w is weight of soil in lb per cu ft.)

and from Eq. 7 the total resisting force against downstream movement is

$$R_d = 654.3 \text{ tons} \times 0.488 + 0$$

$$R_d = 314 \text{ tons}$$

Thus the average over-all factor of safety against downstream shear is

$$F_d = \frac{314}{145.8} = 2.16$$

which, as already indicated, would be considered satisfactory.

[5] This is approximate only. Any suitable method of determining the area under the curve would be satisfactory.

We should now investigate the safety factor at the point of maximum shear. Applying Eq. 2, the average unit shear will be $\frac{145.8}{250} = 0.583$ ton per sq ft and maximum unit shear, Eq. 3, will be $0.583 \times 2 = 1.17$ tons per sq ft.

The point of maximum unit shear is approximately 0.4×250 ft from top of downstream slope.[6] Unit shear strength at point of maximum shear $= 60 \times 120 \times \tan\phi = 60 \times 120 \times 0.488 = 3520$ lb, 1.76 tons sq ft.

$$\text{Factor of safety} = \frac{\text{unit shear strength at this point}}{\text{maximum unit shear}}$$

at point of maximum unit shear $= \frac{1.76}{1.17} = 1.51$, which is satisfactory for point of maximum shear.

7. Safety Against Sudden Drawdown, Upstream Horizontal Shear. *Example.* Still dealing with the dam whose cross-section is shown in Fig. 4, we will proceed to determine the factor of safety of the upstream portion of the dam against sudden drawdown of the water in the reservoir. There is necessarily some riprap on the upstream face which could be very useful in the assumed contingency, but for the present we will neglect its existence.

Assumptions on weights of materials will be as in Art. 6. Applying Eq. 4 of Art. 4, we have for total upstream shear stress with

$$w_s = 134.8 \text{ lb/cu ft}$$

$$H_u = \frac{10{,}000 \times 134.8 \tan^2(45° - \tfrac{1}{2}\phi)}{2} + \frac{62.5 \times 60 \times 60}{2}$$

$$\phi = 26°,\ 45° - \tfrac{1}{2}\phi = 45° - 13° = 32°,\ \tan^2 32° = 0.391$$

$$H_u = 5000 \times 134.8 \times 0.391 + 112{,}600 = 375{,}000 \text{ lb} = 187.5 \text{ tons}$$

Note that complete saturation is assumed, whereas top 10 ft of slope is not actually saturated. This, of course, is on the safe side.

Now to determine the resisting force to the above shear.

$$\text{Area of upstream portion of dam} = \frac{350 \times 100}{2} = 17{,}500 \text{ sq ft}$$

Effective unit weight under sudden drawdown conditions = 72.3 lb/cu ft which is the submerged unit weight of the material.

$$\text{Total effective weight} = 17{,}500 \times 72.3 = 1{,}265{,}000 \text{ lb} = 632.5 \text{ tons}^{7}$$

$$R_u = \text{total shear strength} = 632.5 \times 0.488 = 309.0 \text{ tons}$$

[6] Numerous photoelastic tests indicate this as the approximate point out to which the maximum shear extends.

[7] The fact that the upstream section contains some material above or outside the seepage line is neglected.

Average over-all factor of safety against sudden drawdown

$$F_u = \frac{309}{187.5} = 1.65$$

which is satisfactory. As previously stated, it is generally considered that the average over-all factor of safety against horizontal shear should be at least 2 by this method and accordingly this case requires further investigation by the method illustrated in section 17.

The factor of safety at the point of maximum unit shear in the upstream portion of the dam will now be determined. The average unit shear in accordance with Eq. 5 will be $\frac{187.5}{350} = 0.536$ tons per sq ft, and in accordance with Eq. 6, the maximum unit shear will be twice this or 1.07 tons per sq ft. Location of maximum shear, as before, will be approximately at a point 0.4 of the horizontal distance from the top to the bottom of the slope.

At this point the unit shear resistance will be $60 \times 72.3 \times 0.488 = 2120 =$ 1.060 tons sq ft.

Factor of safety at point of maximum shear $= \frac{1.06}{1.07} = 0.99$.

It is logical to accept a lower factor of safety at point of maximum shear because if the factor of safety at that point were less than unity there would probably be a slight movement and stress would be transferred to other points so long as the over-all factor of safety materially exceeded unity.

8. Discussion of Rough Methods of Slope Design. If as the result of the calculations described in Arts. 3 to 7, inclusive, it is found that the over-all factor of safety for either the downstream or upstream slope is much less than 2, or if the factor of safety at point of maximum shear is much less than 1.5, then the slope should be flattened and the stability calculated again.

In the example chosen we neglected cohesion, but if cohesion is to be taken into account, as it should be if it is significant, a slightly modified value of ϕ in the expression $\tan^2 (45° - \frac{1}{2}\phi)$ may be used. Fig. 5 will be useful in making calculations involving the above quantity as the "equivalent" liquid weight $w_1 = w \tan^2 (45° - \frac{1}{2}\phi)$ may be read directly from the figure. For a method of determining a modified value of ϕ to use in the equation for determining value of w_1, see Art. 9.

The position of the maximum unit shear is empirical as far as these calculations are concerned, but it is in substantial accord with photoelastic determinations and with other methods of calculation. The assumption that the maximum unit shear is twice the average is in accord merely with the simple elastic theory with triangular loading. Photoelastic studies indicate that this is a conservative assumption, as in such model studies the ratio $\frac{\text{maximum unit shear}}{\text{average unit shear}}$ is frequently 1.4.

The foregoing rough method of analyzing stability of slopes of earth dams is claimed to be simple and quick and to give results which agree quite well with results obtained by other suitable methods. It may be applied to any horizontal plane through the dam.

9. Approximate Shearing Stresses in Foundation. The assumption used that an earthen material has an equivalent liquid unit weight which would produce the same shear stress as the material itself has already been used herein. The same assumption or theory has been widely applied to the design of retaining walls. It will now be applied to determining in an approximate manner the shear stress in a foundation.

Foundations consisting largely of coarse sands and gravels or of thoroughly consolidated silts or clays usually show high shear strength, but foundations consisting of fine, loose, cohesionless materials or of unconsolidated clays and silts may be very defective in shear strength and require thorough investigation.

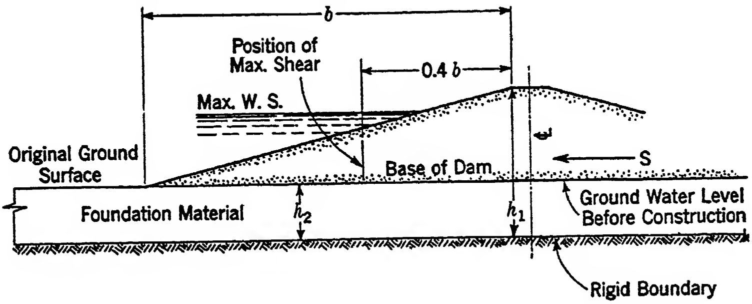


FIG. 6. Approximate shear stress in dam foundation (see Art. 9).

In Fig. 6 there is shown the one-half section of an earth dam founded on a foundation the safety of which is to be determined.

$$S = \frac{(h_1 - h_2)^2}{2} w \tan^2\left(45° - \frac{\phi_1}{2}\right) \qquad [11]$$

S = total horizontal shear down to rigid boundary,

h_1 = vertical distance from top of dam down to the "rigid boundary," such as ledge rock or a sand gravel stratum, the strength of which is great as compared with that of the overlying materials,

h_2 = vertical distance from base of dam (or original ground surface), down to the "rigid boundary,"

b = horizontal distance along base from top shoulder of slope to the toe of the dam,

w = effective weight per cubic foot of the material in its actual condition.

It is here assumed that unit weight of dam and foundation material are the same; if they are different, use a mean (weighted in proportion to depth of each).

ϕ_1 = equivalent angle of internal friction determined as follows:

$$\tan \phi_1 = \frac{c + wh_1 \tan \phi}{wh_1} \qquad [12]$$

in which c is the determined cohesion or no load shear in pounds per square foot and ϕ is angle of internal friction as determined by test. Having determined

$\tan \phi_1$, the value of ϕ_1 in degrees is readily determined from Fig. 5 or from any table of natural tangents.

As S above is the total horizontal foundation shear, the average unit shear is

$$S_a = \frac{S}{b} \qquad [13]$$

in which S_a = average horizontal foundation shear per square foot,
S = total horizontal shear per square foot,
b = horizontal distance along base from top shoulder of slope to the toe of the dam.

The maximum unit shear may be found from the following relationship, which has been substantially checked by photoelastic analyses:

$$S_{\max} = 1.4S_a \qquad [14]$$

in which $S_{\max}$ is maximum shear per square foot in the foundation.

In locating the horizontal portion of the maximum unit shear, its location may be taken at a point $0.4b$ from the upper shoulder of the slope. Photoelastic model studies indicate that this is a fair approximation for its position.

10. Approximate Shear Stress in Foundation. *Example.* In order to provide a numerical example we will now assign values to the various factors of Fig. 6 and Eq. 11 and will then determine the approximate shear stress and later in Art. 11 the factor of safety of the foundation.

The height of the dam in Fig. 6 will be taken as 100 ft.

$$h_1 = 160 \text{ ft}$$

$$h_2 = 60 \text{ ft}$$

$$b = 400 \text{ ft}$$

$$w = 120 \text{ lb}$$

$$\phi = 17°$$

$$\tan \phi = 0.306$$

$$c = 0.20 \text{ ton sq ft} = 400 \text{ lb/sq ft}$$

From Eq. 12

$$\tan \phi_1 = \frac{400 + (160 \times 120 \times 0.306)}{160 \times 120} = 0.326$$

and from Fig. 5, $\phi_1 = 18°$.

Also from Fig. 5, the value of $w \tan^2 \left(45° - \frac{\phi}{2}\right)$ for $w = 120$ lb per cu ft and $\tan \phi_1 = 0.326$ is found as 63.5 lb (equivalent liquid unit weight). Then from Eq. 11 total shear in foundation along b (Fig. 6)

$$S = \frac{160^2 - 60^2}{2} \times 63.5 = 698{,}500 \text{ lb} = 349.3 \text{ tons}$$

and from Eq. 13 the average unit shear in the foundation

$$S_a = \frac{349.3}{400} = 0.875 \text{ ton per sq ft}$$

In accordance with Eq. 14 the maximum unit shear will be $1.4 \times 0.875 = 1.23$ tons per sq ft.

11. Approximate Factor of Safety Against Foundation Shear. *Example.* In this section we are dealing with the factor of safety at the time of completion of the dam before headwater is raised. It is to be understood that ϕ and c given in Art. 10 apply to the foundation at the state of consolidation that it had at that time.

Continuing the same example as in Art. 10, we will now determine the factor of safety against shear in the foundation at the time the dam was just completed.

Unit shear strength of foundation below toe $= c + w_1h_2 \tan \phi$, where w_1 is effective unit weight of material. Assuming that ground-water level is at ground surface in this case, the submerged unit weight $= 120 - 62.5 = 57.5$ lb per cu ft. Unit shear strength below toe $= 400 + (60 \times 57.5 \times 0.306) = 1455$ lb $= 0.73$ ton per sq ft.

At point in foundation under upper shoulder of slope

$$w_1 = \frac{60 \times 57.5 + 100 \times 120}{160} = 96.5 \text{ lb per cu ft}$$

Shear strength at this point $= 400 + (160 \times 96.5 \times 0.306) = 5140$ lb $= 2.57$ tons per sq ft

Average unit shear strength $= \dfrac{0.73 + 2.57}{2} = 1.65$ tons per sq ft.

Over-all average factor of safety against foundation shear $= \dfrac{1.65}{0.875} = 1.9.$

(0.875 ton per sq ft is average unit shear in foundation from Art. 10.)

We must also investigate the factor of safety in the foundation against shear at point of maximum unit shear located $0.4b$ from upper shoulder (see Fig. 6). The mean effective unit weight w_1 will be $\dfrac{57.5 + 120}{2} = 88.8$ lb per cu ft. This is because at this point there is 60 ft below and 60 ft of material above the ground-water level.

Unit shear strength at point of maximum shear $= c + wh \tan \phi$, $400 + (120 \times 88.8 \times 0.306) = 3670$ lb $= 1.84$ ton.

Factor of safety against shear at point of maximum shear $= \dfrac{1.84}{1.23} = 1.5$ which is satisfactory for the point of maximum unit shear.

It should be understood that it would be possible to have a factor of safety of less than 1.0 at point of maximum unit shear. In that case there would be local

movement at this point, but if the average factor of safety exceeded unity, the resisting forces would be mobilized and failure would not occur.

The computations just given in this section apply to conditions with the dam just completed but the headwater not raised. When the headwater is raised a new condition will exist which must usually be also investigated in connection with the design analyses. Thus for the downstream portion of dam and foundation where shear strength ($c + w_1 h \tan \phi$) all material below seepage line should be taken at submerged unit weight. Also when investigating safety of upstream portion of dam after raising headwater, the assumption of sudden drawdown should be used. Utilizing the principles of Arts. 3 to 7 and 10 and 11, one may readily make the analyses to determine safety of foundation on the assumption of the headwater having been raised.

In many cases the time when the dam is just completed, as in Arts. 10 and 11, is the most critical for the safety of the foundation.

This is for the reason that by the time the headwater can be raised and a sudden drawdown takes place, the foundation will have received a material additional amount of consolidation, resulting in a higher value of the angle of internal friction, ϕ, and an increased cohesion, c.

12. Elastic Theory for Determining Shear in Foundation. The elastic theory has been utilized by Terzaghi,[8] Jürgenson,[9] Carothers,[10] Timoshenko,[11] and others for the solution of problems involving stress in foundations. With the elastic theory there is a fairly definite relationship between stress and strain (movement), and when a soil is in the elastic state, the stress continues to increase with increasing strain. On the other hand, when a soil is in a plastic state shearing stresses have reached the shearing strength and the strain (movement) increases with the stress remaining constant.

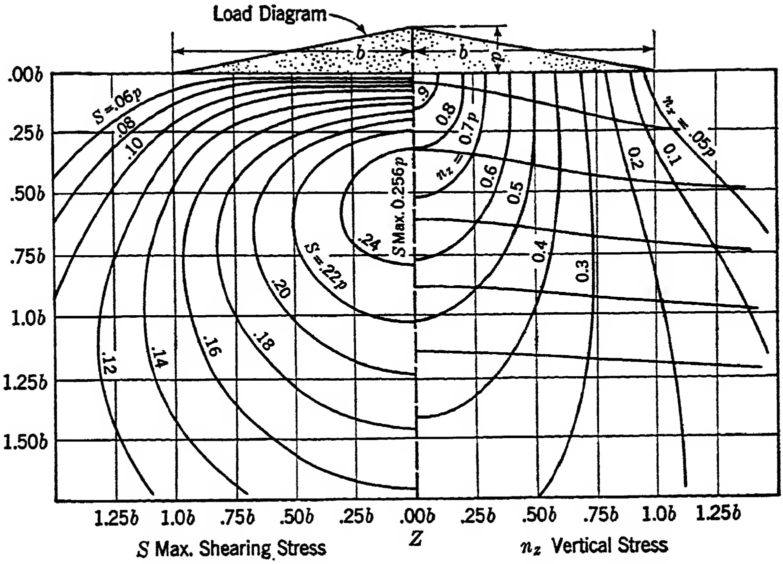
In Fig. 7 is given the distribution of shearing stresses in a foundation under the slope of an embankment. Jürgenson assumed a homogeneous isotropic material of infinite depth in making the computations for this figure. The maximum load per square foot here extends for an indefinite distance, whereas with an earth dam the maximum load per square foot extends for a relatively short distance before it reaches the top of the opposite slope. The reason for using the so-called terrace loading is that this loading gives more nearly the true horizontal position of the maximum unit shear stress than the triangular loading does. Thus for triangular loading maximum unit shear is at dam center line, whereas for terrace loading it is at the midpoint of the slope (= ½ horizontal distance shoulder to toe). Photoelastic tests give a similar position (0.4 to 0.55). Thus it is believed that conditions in Fig. 7 are quite analogous to the conditions in the foundations of many earth dams.

[8] KARL TERZAGHI, *Erdbaumechanik*, Franz Deuticke, Vienna, 1925.

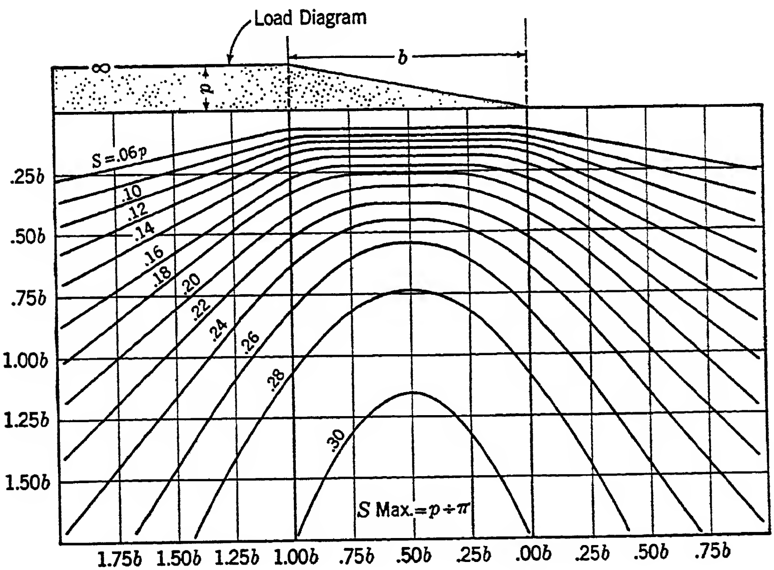
[9] LEO JÜRGENSON, "The Application of Theories of Elasticity and Plasticity to Foundation Problems," *J. Boston Soc. Civil Engrs.*, July 1934.

[10] S. D. CAROTHERS, "Direct Determination of Stresses," *Proc. Roy. Soc.* (London), Ser. A, Vol. 97, p. 110.

[11] S. TIMOSHENKO, *Theory of Elasticity*, McGraw-Hill Book Co., New York.



(a)



(b)

FIG. 7. Distribution of maximum shearing stresses in foundation of homogeneous isotropic material of infinite extent. (*From "The Application of Theories of Elasticity and Plasticity to Foundation Problems," by Leo Jürgenson, J. Boston Soc. Civil Engrs., July 1934.*)

In Fig. 7, b = the horizontal distance from the top shoulder of the slope to the toe in feet,
P = load per square foot on foundation at center line of dam,
$\pi = 3.1416$,
S = shear per square foot.

It is desired to know the factor of safety of the deep homogeneous isotropic foundation of a 100-ft earth dam. The foundation material is a fine silt without cohesion (see Fig. 7). Use plane at depth below foundation level $= 0.5b$ for obtaining a factor of safety. Factor of safety at other depths should also be determined.

Other assumptions are:

$\tan \phi = 0.4$,
Unit effective weight of material is 120 lb per cu ft,
Height of dam = 100 ft,
Slope of face is 1 on 2, or $b = 200$ ft.

Referring to Fig. 7, it is found that at depth $= 0.5b$ at the quarter point the unit shear in the foundation is $0.25P$ or in this case

$$S = 0.25P = \frac{0.25 \times 100 \times 120}{2000} = 1.5 \text{ tons per sq ft}$$

In determining the unit shear strength at this point we will use the total depth of material at the quarter point. Shear strength = total depth at midpoint of slope $\times$ unit weight $\times \tan \phi = \dfrac{150 \times 120 \times 0.4}{2000} = 3.6$ tons per sq ft.

Factor of safety at midpoint of slope and at depth in foundation equal to $0.5b = \dfrac{3.6}{1.5} = 2.26$.

Similarly at foundation depth of $0.25b$ for the above, unit shear stress would be $0.18P$ or $S = \dfrac{0.18 \times 100 \times 120}{2000} = 1.08$ tons per sq ft, and unit shear strength $\dfrac{100 \times 120 \times 0.4}{2000} = 2.4$ tons per sq ft.

Factor of safety at quarter point and at depth in foundation is equal to $0.25b = \dfrac{2.4}{1.08} = 2.22$.

Several similar calculations are usually necessary in order to determine the minimum factor of safety of the foundation. In applying this method, it should be borne in mind that it is directly applicable only to a homogeneous, isotropic foundation of infinite depth.

13. Shear in Plastic Foundation—Jürgenson Formula. Jürgenson [12] derived a very simple formula for obtaining the approximate shear stress in a plastic layer in the foundation of an earth dam of triangular cross-section.

$$s = \frac{Pa}{L} \qquad [15]$$

in which s = maximum unit shear,
P = the maximum unit pressure on the foundation at the plastic layer,
a = ½ the thickness of the plastic stratum,
L = ½ the base width of the structure.

The formula has been used a great deal because of its simplicity. It is directly applicable, of course, only to the conditions which Jürgenson assumed. See Art. 12 for definition of plastic state.

Although the formula may in some cases give results which are quite far from the truth, it is believed to be frequently suitable for use in preliminary analyses. In the authors' opinion the Jürgenson formula should not be applied to thick plastic layers and certainly not to layers thicker than $\frac{1}{2}L$. For a thorough and final analysis of a plastic foundation photoelastic methods are desirable.

In using the Jürgenson formula it should be understood in general to portray a condition after the material has been overstressed. The material at that time having passed from the elastic state to the plastic state, movement will continue with the stress remaining constant. (See also Art. 12.) Under this condition the unit stress throughout the plastic layer is the same.

14. Swedish Geophysical Method of Stability Analysis. K. E. Petterson first applied the circle method to the analysis of a soil failure in connection with the failure of a quarry wall in Göteborg, Sweden, in 1916. A Swedish National Commission, after studying a large number of failures, published a report in 1922 showing that the line of failure of most such slides roughly approached the circumference of a circle.[13]

As shown by the investigation, the failure circle might pass above the toe, through the toe, or below it. By investigating the strength along the arc of a large number of such circles, it is possible to locate the circle which gives the lowest resistance to shear. This general method has been quite widely accepted as offering an approximately correct solution for the determination of the factor of safety of the slope of an embankment and of its foundation.

An early difficulty in utilizing the method was the very large number of possible failure circles which it was necessary to analyze. Developments in the method of analysis have been made by W. Fellenius, Terzaghi, Gilboy, Casagrande, Taylor, and others, with the result that the satisfactory analysis of the stability of slopes, embankments, and foundations by means of a dangerous circle method is no longer an unduly tedious procedure.

Essentially the method consists in passing a circle through the slope, or through the slope and foundation, taking moments of all the forces about the center acting

[12] LEO JÜRGENSON in *J. Boston Soc. Civil Engrs.*, July 1934.
[13] Statens Järnvagärs Geoteknisca Commission Slutbetankanda, 31 May, 1922.

along the circumference of the circle. As all the forces have the radius of the circle for their lever arm, the radius cancels out of the computations. Consequently, to get the factor of safety, all that is necessary is to add up all the resisting forces along the circumference of the circle and divide this by the sum of all the forces tending to produce movement along the circumference of the circle.

15. Dangerous Circle Analysis with Failure Plane Through Toe. In this section a procedure for minimizing the amount of cutting and trying required to locate the most dangerous circle will be given.

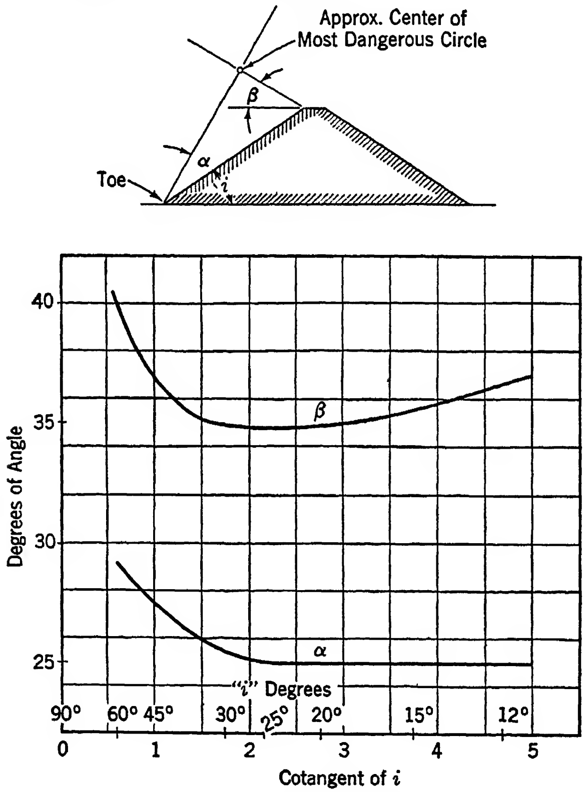


FIG. 8. Angles α and β for different slopes to determine center of most dangerous circle which passes through toe (see Art. 15).

Knowing the proposed slope of the upstream or downstream face for an earth dam, refer to Fig. 8, using the cotangent of the slope angle, i, as argument and pick off the corresponding values of the angles of α and β.

Thus in Fig. 9, the proposed slope of the face of the dam is 1 on 3 and cot is 3.0. Using cot = 3.0, refer to Fig. 8, it is found that i is 18.43° (or 18° 26′).

$$\alpha \text{ is } 25°\ 00'$$

$$\beta \text{ is } 35°\ 00'$$

Thus in Fig. 9 α is laid off from the toe of the slope and β is laid off from a horizontal at the top of the slope. Then the two outer sides of the angles will intersect in some point, O. With O as the center, inscribe a circle having AO as radius, cutting the embankment lines. The area intercepted between the circumference of the circle and the lines of the embankment is then divided up into a number of vertical slices as is indicated in Fig. 9. The vertical slices are preferably of equal width. The number of slices used should not be less than five, and it is seldom necessary to use more than twelve. The area and effective weight of each of the vertical slices is determined, and a vertical line proportional to the

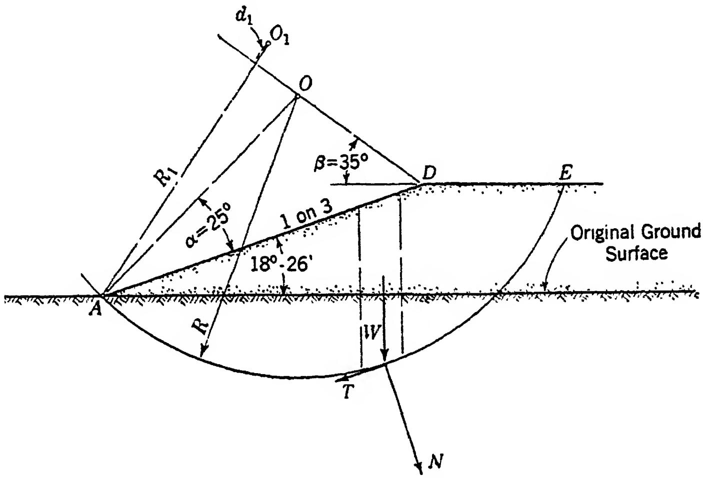


FIG. 9. Method of analysis when dangerous circle passes through toe (see Art. 15).

total effective weight of the slice is drawn from its center of gravity. At the circumference this weight is resolved into its normal and tangential components. This process is followed for each of the vertical slices.

The next step is to add up all the tangential forces or T forces for the slices (with proper regard to sign). Also add up all the normal forces.

ΣT = force tending to produce movement along the circumference (shear). The force tending to prevent movement is

$$\Sigma N \tan \phi + Lc \tag{16}$$

which is the total shear strength of the material along the arc of the circumference $\overline{AE}$.

In the above

ΣT is summation of tangential forces tending to produce movement,
ΣN is summation of the normal forces for all of the slices,
ϕ is angle of internal friction of the material as determined by tests,
L is length of arc $\overline{AE}$ intersecting the embankment (see Fig. 9),
c is cohesion per square foot,

F_s is the factor of safety against shear failure along the arc of the circle under investigation.

$$F_s = \frac{\Sigma N \tan \phi + Lc}{\Sigma T} \qquad [17]$$

If a satisfactory factor of safety is shown it means merely that shear failure need not be expected along the assumed circle, but it may occur elsewhere along some other circle unless the one under investigation is already known to be the most dangerous. Consequently a number of circles must usually be drawn and analyzed.

For the center of additional trial circles through the toe of slope, proceed as follows: Back up the line OD, Fig. 9, and then up and away from it in a perpendicular direction in such a way that Od_1 is not greater than $\frac{1}{2}OD$ and O_1d_1 is approximately equal to $\frac{1}{3}Od_1$. O_1 will then be the center of a new trial dangerous circle with radius R_1.

This new circle is analyzed as before and its factor of safety determined. The procedure is continued until the circle is found which gives the lowest factor of safety.

For the given conditions this last circle is the most dangerous which passes through the toe of the slope. Provided the foundation of the dam has the same or a lesser shear strength than the dam itself, it is still possible that there is another circle which does not pass through the toe of the slope which is still more dangerous. In choosing centers for trial circles it is not advisable to adhere too closely to rules. Art. 16 suggests a procedure which will aid in locating the center of such a circle.

16. Dangerous Circle Analysis With Failure Plane Below Toe. Theoretically if the materials of the dam and foundation are entirely homogeneous, any practicable earth dam slope may have its most dangerous failure plane below the toe

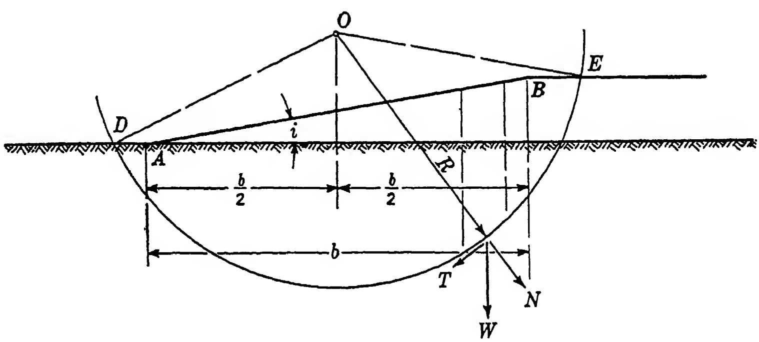


FIG. 10. Method of analysis when dangerous circle passes below toe (see Art. 16).

of the slope. Fellenius found that the angle intersected at O in Fig. 10 for this case is about $133\frac{1}{2}°$. To find the most dangerous circle below the toe, it is suggested that the procedure be as follows:

If the slope of the face of the dam is as shown in Fig. 10, erect a vertical at the midpoint of the slope. On this vertical will lie the center O of the first trial dangerous circle. In locating the trial circle use an angle of $133\frac{1}{2}°$ between the two radii at which the circle intersects the surface of the embankment and foundation. After the first trial dangerous circle has been drawn, the method of analysis is the same as that described in Arts. 14 and 15.

After the first dangerous circle has been analyzed the trial center is moved somewhat to the left, the radius shortened, and a new trial circle drawn and analyzed as in Art. 14, additional centers for the trial dangerous circle are spotted, and the resulting circles analyzed as described in Art. 14. Finally after a number of circles have been analyzed the computed factors of safety of the various circles are tabulated and compared. The circle showing the lowest factor of safety will be the most dangerous circle.

While the methods described apply directly only to entirely homogeneous materials, they may also be applied with a sufficient degree of accuracy to dams and foundations which are not homogeneous provided that sound judgment is utilized.

17. Example of Dangerous Circle Analysis by Slices. In Fig. 11 is given a simple example of the analysis of the downstream slope of an earth dam without any saturation. The structure is homogeneous and the material is a silt having an effective weight of 110 lb per cu ft, the dam is 62.5 ft high, and the slope is 1 on 3. The angle of internal friction (ϕ) for the material is 30°, giving a $\tan \phi = 0.58$. There is also a little cohesion amounting to 50 lb per sq ft.

The center of the dangerous circle in Fig. 11 has been located by the methods outlined in Arts. 15 and 16. The area above the circle has been divided into a number of slices. It is not essential that these slices should be of equal width, but, as in Fig. 11, it is often convenient to make them so. A width of 20 ft was chosen for each of the nine slices in this case. For instance in slice six, the weight of the slice is $28 \times 20 \times 110 = 6160$ lb $= 6.16$ kips. This weight is considered as suspended at the center of gravity of the slice and may be resolved into its components T, the tangential force, and N, the normal force. Where the material is homogeneous, as in this example, the weight of each slice is directly proportional to the depth of the slice. We can then facilitate calculations as indicated in the table of Fig. 11 by determining the N and T components from the depth D; and after we have found the sum of these N and T, we can multiply by width of section and weight to get the total N and T forces.

The forces tending to resist movement are obtained by summing up all the N forces, multiplying them by $\tan \phi$, and adding the cohesion (if any) along the arc of the circle. The forces tending to produce movement are the T forces. These are summed up as indicated in Fig. 11, and the factor of safety is determined as 1.96. In cases where several different materials are included and particularly where drawdown conditions must be considered, these analyses are more complicated.

Fig. 11 applies to the downstream portion of an earth dam without any saturation. If there is saturation, alter the procedure as follows:

Analysis			
Section	D Depth of Section	Components of D	
		T	N
1	5.0	−0.8	5.0
2	15.0	−1.0	15.0
3	21.5	+1.2	21.4
4	25.8	+4.2	25.3
5	28.0	+7.4	27.0
6	28.0	+11.0	26.0
7	25.0	+12.0	22.0
8	18.5	+11.5	15.3
9	7.0	+4.7	5.0
		$\Sigma T = 50.2$	$\Sigma N = 162$

Weight of Slice $= \Sigma D \times 20 \times 110^{\#}$

ΣT Forces $= \Sigma T$ Components $\times 20 \times 110^{\#}$
$= 50.2 \times 20 \times 110^{\#} = 110.5^{k}$

ΣN Forces $= \Sigma N$ Components $\times 20 \times 110^{\#}$
$= 162 \times 20 \times 110^{\#} = 356^{k}$

$$\text{Factor of Safety} = \frac{\Sigma N \tan\phi + \text{Cohesion}}{\Sigma T}$$
$$\frac{356^{k} \times .58 + 10^{k}}{110.5} = \frac{216.5}{110.5} = 1.96$$

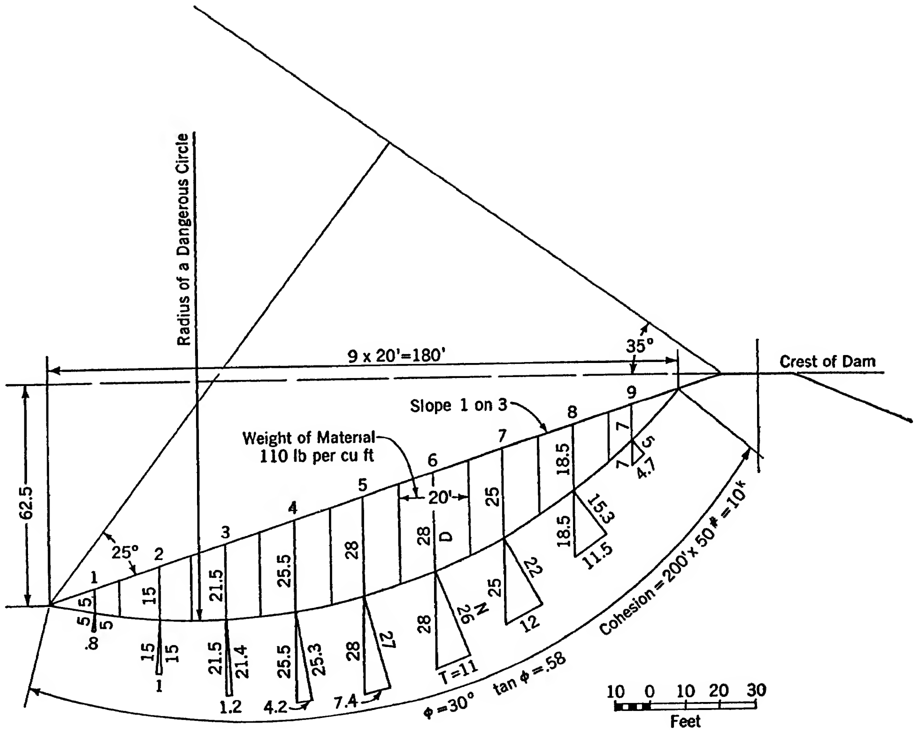


FIG. 11. Example of dangerous circle analysis by slices (see Art. 17).

1. Locate the seepage line by methods of Arts. 10 to 18, of Chapter 17.

2. In obtaining driving forces use saturated unit weights for materials below the seepage line and dry or moist unit weights for materials above the seepage line.

3. In obtaining resisting forces use buoyant weights for materials below the seepage line and dry or moist unit weights for materials above the seepage line.

18. Taylor's Stability Numbers. If the slope angle, height of fill, effective weight of material per cubic foot, angle of internal friction, and unit cohesion are

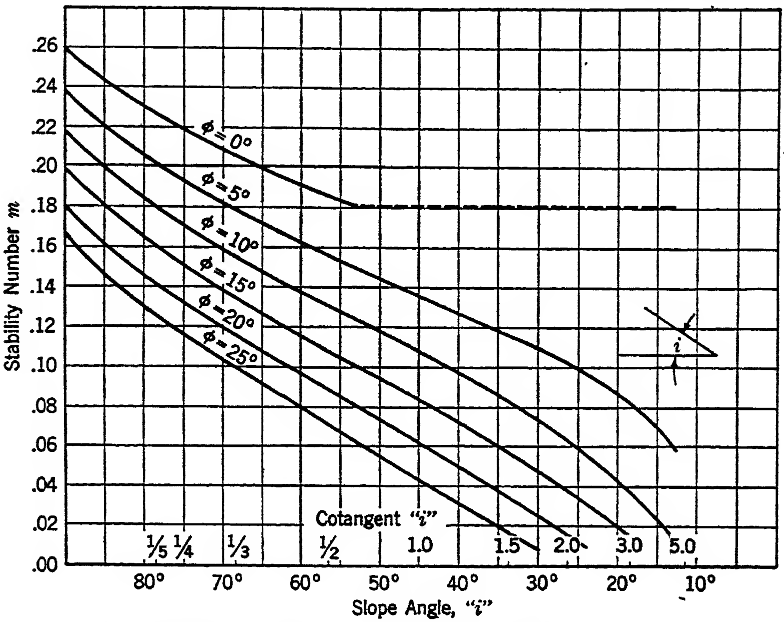


FIG. 12. Taylor's stability numbers for various slopes and angles of internal friction (ϕ) (see Art. 18).

Note. These curves are for circles passing through the toe, although for values of "i" of less than 53°, it has been found that the most dangerous circle passes below the toe. However, these curves may be used without serious error for slopes down to 1 on 4 ($i = 14°$).

(*Adopted from "Stability of Earth Slopes," by Donald W. Taylor, p. 213, J. Boston Soc. Civil Engrs., July, 1937.*)

known, the factor of safety may be determined. In order to make unnecessary the more or less tedious stability determinations, Taylor [14] conceived the idea of determining the stability of a large number of slopes throughout a wide range of slope angles and angles of internal friction and representing the result by an abstract number which he calls "the stability number."

$$m = \frac{c}{FwH} \qquad [18]$$

[14] DONALD W. TAYLOR, "Stability of Earth Slopes," *J. Boston Soc. Engrs.*, Vol. 24, July 1937, p. 197.

in which m = Taylor's stability number.
c = cohesion in pounds per square foot,
F = factor of safety,
w = effective weight of material in pounds per cubic foot,
H = height of slope in feet.

Transposing:

$$F = \frac{c}{mwH} \tag{19}$$

In order to illustrate the use to which Eq. 19 may be put to help solve the problems of design, we will assume the following: The factor of safety of the downstream slope is desired.

Slope, 1 on $2\frac{1}{2}$,
ϕ, angle of internal friction, 15°,
c, cohesion, 750 lb per sq ft,
w, effective weight per cubic foot of material, 120 lb
H = 140 ft.

Using Fig. 12, we find for the above that m, the stability factor will be = 0.028 for a slope of 1 on $2\frac{1}{2}$, and $\phi = 15°$. Then for Eq. 19

$$F = \frac{750}{0.028 \times 120 \times 140} = 1.6$$

which is a satisfactory factor of safety under most conditions. It will be noted that, other things remaining the same, the less the effective unit weight of the material, the greater the factor of safety according to the formula. As it is evident that saturation decreases the factor of safety, caution is necessary in the application of this formula.

19. Abbreviated Method of Dangerous Circle Analysis. When there are several dangerous circles to be analyzed, the usual procedure by the slice method is quite tedious. Accordingly, N. C. Courtney has developed an abbreviated method of graphical solution which requires only a small part of the time otherwise required for analysis by the method of slices and gives results just as precise.

Fig. 13 shows a section through the downstream slope of a dam and illustrates the application of this method. A dangerous circle has been drawn cutting the slope as indicated. Any vertical line from the slope of the dam to the dangerous circle represents the weight (W) of a strip infinitely small in width. The components N and T of one of these vertical lines represent the resolution of the weight W into forces normal and tangential to the dangerous circle. If we now plot these N and T components from a horizontal base for various points throughout the section and join their extremities, we obtain curves, the areas under which represent respectively the summation of all N and T components.

These areas, determined by planimetering or otherwise, multiplied by the effective unit weight of material give the total N and T forces acting on the particular circle. The summation of the N forces when multiplied by the tangent of the angle of friction gives the total resisting frictional force along the arc. Any

cohesive value of the material along the length of the arc of the dangerous circle must be added to the friction value determined from the N forces to obtain the total force resisting shear along the arc. The summation of the T forces gives the total driving force tending to cause the material to shear along the arc.

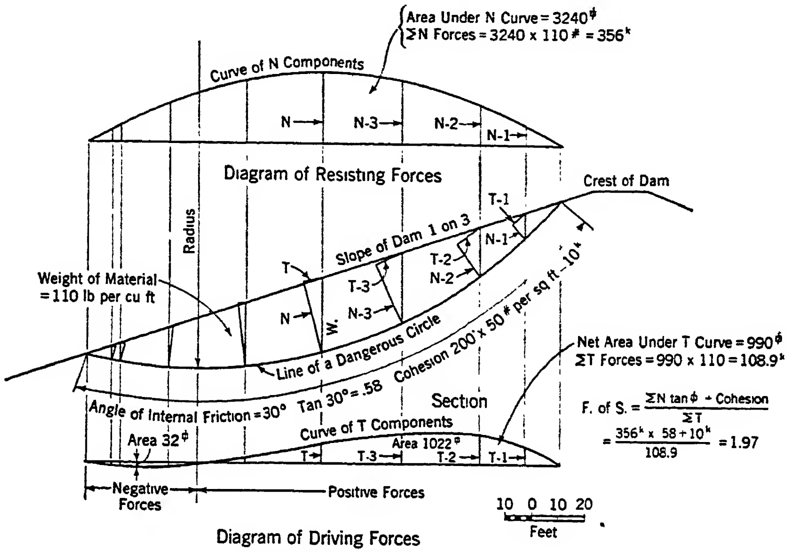


FIG. 13. Dangerous circle analysis, abbreviated method (see Art. 19).

The factor of safety for any given dangerous circle is obtained, as in the slice method, from the formula

$$F_s = \frac{\Sigma N \tan \phi + Lc}{\Sigma T} \qquad [17]$$

in which F_s = factor of safety against shear along the arc of the given circle,
ΣN = summation of normal forces,
= area under the N curves multiplied by the unit effective weight of the material,
$\tan \phi$ = tangent of angle of internal friction of the material which the circle cuts,
L = length of arc of circle,
c = unit cohesion as pounds per square foot of the material cut by the circle,
ΣT = summation of tangential forces,
= area under T curve times unit effective weight of material.

This method is readily adapted to any complicated case, as the N and T curves can easily be plotted for various materials, slopes, and conditions. The application of the method to a simple case is thus shown in Fig. 13. In this figure the dam is the same as in Fig. 11 in every respect. The only difference between the

two figures is in the method of analysis. It is worth noting that the factor of safety by the abbreviated method is 1.97 (Fig. 13), whereas by the slice method, Fig. 11, it is 1.96.

20. Analysis of Upstream Slope—Drawdown Conditions. The upstream slope of a dam is subject to water pressure under normal conditions. When sudden drawdown takes place this pressure is removed above the drawdown level. The shell material is ordinarily submerged below pond level, but immediately after drawdown the shell material is either moist, owing to being able to drain as rapidly as the pond level goes down,[15] or it is saturated, owing to the inability of the shell to drain freely.

When determining the stability of an upstream dam slope under full head water conditions, the resisting and driving forces acting on any dangerous circle are calculated on the basis of all the materials being submerged except those above the normal pond level.

When determining the stability of an upstream dam slope against drawdown, we must determine if the shell material is or is not sufficiently free draining.[15] If the shell material is clean rock or coarse gravel and, therefore, will drain as fast as the pond can go down, then the resisting and driving forces within the drawdown range are calculated for the dry or moist weight of the rock material.

If the shell material within the drawdown range will not drain as rapidly as the pond can be drawn down, the resisting forces are calculated for the submerged weight of the material below water surface and the dry weight above water surface but the driving forces are calculated for the saturated weight of the shell material below water surface plus dry weight above water surface. All materials below the drawdown level are submerged, and all resisting and driving forces below the drawdown level are calculated on the basis of the submerged weight of the materials.

Fig. 14 shows an example of an upstream slope with a rock shell and impervious core that has been suddenly drawn down from a normal pool at elevation 90 to elevation 40.

Curves enclosing N and T areas are plotted as described in the previous article. The area under these curves are divided to show the N and T areas which are either above, within, or below the drawdown level as well as for the rock shell and impervious material within these respective levels. In the illustration shown the shell is rock and free-draining. The resisting forces are then calculated on the dry weight for the rock material above the normal water level and within the drawdown level. Detailed calculations are given to show the method of determining the factor of safety for this case. If the shell material had not been sufficiently free draining so that it would drain as fast as the pond could go down, the submerged weight would have been used to calculate the resisting forces.

As indicated, the slope of the impervious section is 1 on 1.5 and the outside slope is 1 on 2.5. Attention is called to the fact that the outside shell with a

[15] Only relatively clean rock with large interstices would fully meet this requirement as, for instance, ordinary riprap.

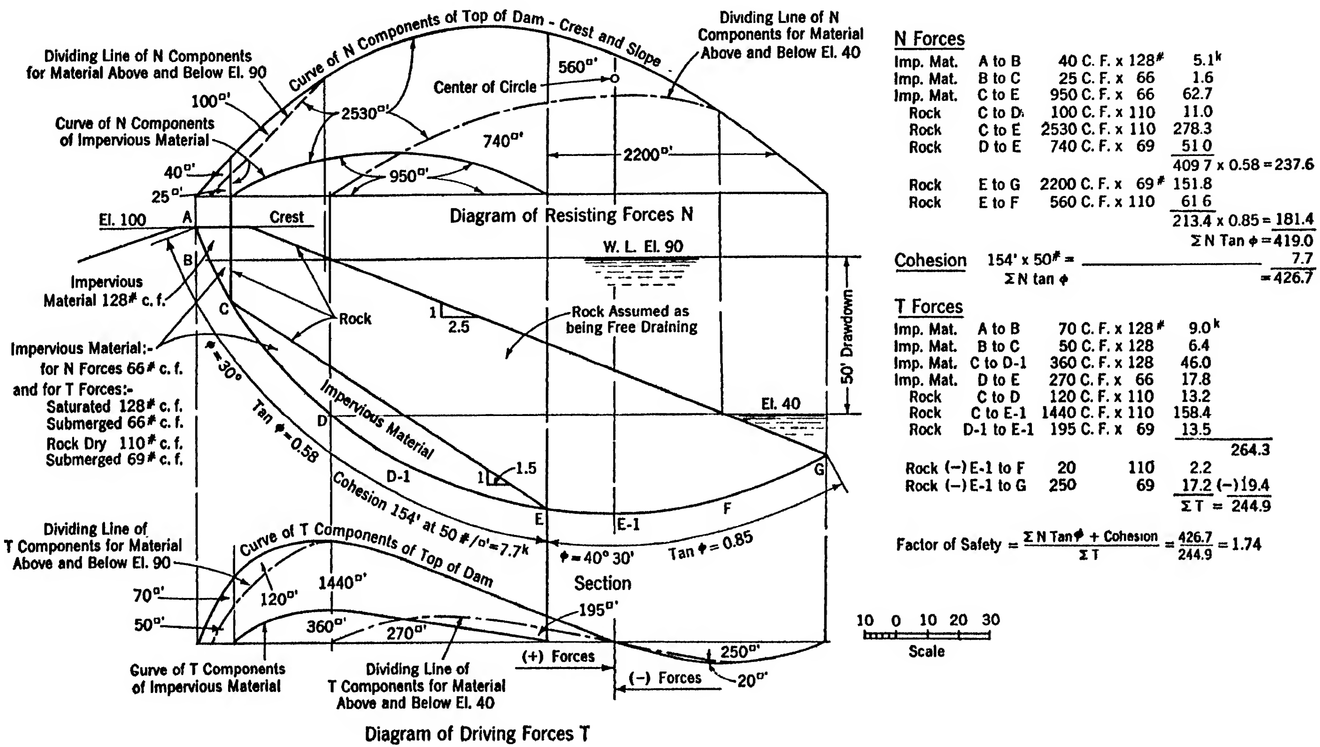


FIG. 14. Dangerous circle analysis, nonhomogeneous dam with drawdown (see Art. 20).

slope of 1 on 2.5 is composed of a relatively clean dumped rock fill, i.e., any fine soil such as sand or dirt is far too small in amount to fill the interstices in the rock fill. This fact permits the shell to drain just as rapidly as it is possible for the water surface of the pond to go down, and hence in all calculations involving the rock shell above the drawdown level (elevation 40) the dry or moist unit weight of the rock fill may be used.

Full data relating to the materials and the summation of the acting and resisting forces are given in Fig. 14, and the factor of safety for the given conditions is worked out as 1.74.

21. Stability of Hydraulic Fill Dams. Hydraulic fill dams, constructed by transporting, sorting, and depositing the material by the agency of water, have been used in many places where materials available are suitable and where local conditions favor economical construction.

Fig. 15 shows a typical hydraulic fill dam. The central portion of the dam or core is composed of silty clay and the outer portions of the dam or shells are composed of sands and gravels (see Art. 36, Chapter 19). Owing to the process of construction peculiar to hydraulic fill dams (Arts. 34 and 35, Chapter 19), the

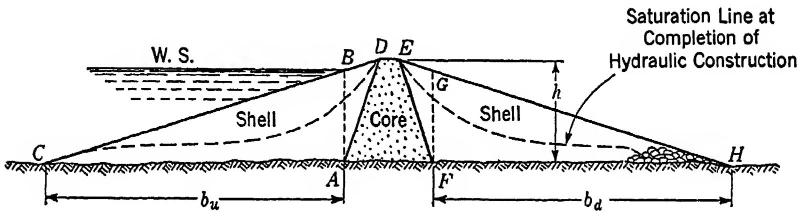


FIG. 15. Typical hydraulic fill dam.

coarse material is segregated and deposited in the shells and the fine materials (clays and silts) are deposited in the core near the center of the dam. The materials in the core, being much finer than those in the shell, stay in suspension longer and settle more slowly.

Thus the mixture of water and core material is, to some extent, when first delivered, a liquid having a greater unit weight than water. Immediately after delivery to the core pool, however, the material begins to settle out. If excessively fine and particularly if some of it consists of colloids, the material will continue in suspension for a very long time. (See Art. 3, Chapter 16.) On the other hand if the fine material contains a high percentage of coarse silts and fine sands, the material may settle out quite promptly and the core may then rapidly become stable.

The shells of sand, gravel, and stone become stable and strong at once when deposited, but the core, as indicated above, is not stable at once, and if very fine, it may not become thoroughly consolidated for many years after it is placed.

Thus the core when first placed and generally for a long time thereafter is in a more or less liquid condition and exerts a substantial pressure on the shells of the dam tending to force them outward away from the core. Frequently the core of a hydraulic fill dam is spoken of as being a semi-liquid.

The pressure exerted by the core on the shells may be conceived of as equivalent to that which would be exerted by a liquid having a certain weight per cubic foot. This concept has already been used herein. (See Arts. 3 and 4 and 7 to 11 of this Chapter.)

According to this concept equivalent liquid weight per cubic foot $= w \tan^2 (45° - \frac{1}{2}\phi)$, in which w = unit weight of the material.

In Fig. 16 it will be noted that the outside slopes are 1 on 3 and the slope of the core is 3 on 1. The slopes as shown are frequently satisfactory slopes for a hydraulic fill dam.

If, however, the core had a slope of 1 on 2, it would mean that the quantity of materials in the shells would be tremendously reduced and that in all probability the shells would not be able to resist the semi-liquid pressure of the core.

Efforts have been made to determine the actual horizontal pressure exerted in the shell by the core. Pressure cell measurements have indicated pressures ranging from about 20 lb per sq in. up to more than 75 lb per sq in. Indiscriminate faith should not be placed in such observations, however, as some pressure cells have given quite erratic records.

At Germantown Dam, Ohio, in one instance, Goldbeck pressure cells indicated a lateral pressure equal to 23 lb per sq in. at a depth of 57 ft, or that which would be produced by a liquid weighing 58.3 lb per cu ft. Also at Taylorsville Dam, Ohio, Goldbeck pressure cells, in one instance, showed a lateral pressure of 23 lb per sq in. at a depth of 71 ft, equivalent to that due to a liquid weighing 47 lb per cu ft.[16]

At Fort Peck, Mont., it was found that for pressure cell readings at a depth of 61.8 ft, the lateral pressure was equivalent to that of a liquid weighing 71 lb per cu ft.

At Kingsley Dam, near North Platte, Nebr. (Art. 42, Chapter 19), where there was a considerable number of soil pressure and water pressure cells installed, W. J. Turnbull found that "the mean ratio of lateral pressure to estimated vertical loads for the core material indicate that, in general, one could expect less than 45 per cent of the vertical load in lateral pressure." In other words, under the conditions at Kingsley Dam with unit weight of core 100 lb per cu ft, the equivalent liquid unit weight of the core material would not be over 45 lb per cu ft. This applies only to dams like Kingsley, a dam where the core is a very stable loess. For fine clay the ratio of lateral to vertical load may be as high as 0.7 during construction.

The authors believe that for clays and clayish silts (effective size 0.005 mm or less) containing appreciable percentages of colloids the average equivalent liquid weight of core material on completion of the hydraulic fill may be as high as 80 lb per cu ft. For silts, coarse clays, and rock flour, effective size 0.01 mm or more, the average equivalent liquid weight may be as low as 30 lb per cu ft.

For specific cases where tests are available on the value of ϕ the equivalent liquid weight may be obtained from Fig. 5 or from the Rankine formula. In general, tests show that the angle of internal friction, ϕ, of core material in Eq. 21

[16] See CHARLES H. PAUL, *Trans. Am. Soc. Civil Engrs.*, 1922, p. 1181.

will range from about 15° for a fine clay core up to 30° for a stable core of rock flour or coarse silt at the critical period when the construction of the dam is being completed.

22. Gilboy Hydraulic Fill Dam Formula. Gilboy [17] presents the following formula for the stability of hydraulic fill dams on the assumption of a fully liquid core.

$$\sqrt{R} = \frac{(C - A)\sqrt{1 + B^2} + \sqrt{C - A}\sqrt{C - B}\sqrt{1 + A^2}}{(1 + C^2) - (C - A)(C - B)} \qquad [20]$$

in which A = cotangent of angle of core slope with horizontal,
B = cotangent of angle of internal friction of shell material,
C = cotangent of angle of outer slope with horizontal,
R = ratio of unit weight of core to unit weight of shell.

Manifestly the factor of safety for the above formula is represented by the degree to which the actual core departs from a condition of liquidity.

23. Factor of Safety with Gilboy Formula. In order that the Gilboy method may give directly a factor of safety against failure, an additional ratio will be introduced.

R_1 = ratio of equivalent unit liquid weight of core material to the effective unit weight of shell material.[18]

$$R_1 = \frac{w_1 \tan^2 (45° - \phi_1/2)}{w} \qquad [21]$$

in which w_1 = unit weight of core material in actual wet or moist condition,
ϕ_1 = angle of internal friction of core,
w = effective unit weight of shell material,
R = ratio as defined in Art. 22,
$F_s = \dfrac{R}{R_1}$, in which F_s is factor of safety of the shells against shear from the internal pressure of the core.

24. Safety of Shells Against Horizontal Shear. One may also wish to determine approximately the safety of the hydraulic fill dam against horizontal shear. Thus in Fig. 16 the force tending to produce shear would be that due to the lateral pressure from the semi-liquid core just at the moment that the hydraulic fill is completed. Hydraulic fill dams typically have a lower factor of safety during construction than at any later time because thereafter the cores gradually consolidate with time and exert less and less pressure on the shells.

Resisting the pressure from the core is the shear resistance of the shells, assuming that the horizontal plane is considered cut through shells and core. A por-

[17] GLENNON GILBOY, "Mechanics of Hydraulic Fill Dams," *J. Boston Soc. Civil Engrs.*, Vol. 20, No. 3, July 1934, p. 185.

[18] Thus in a computation for stability during construction, the unit weight of shell utilized might be weighted mean between the submerged unit weight and dry or moist unit weight.

tion of both shells, as shown in both Fig. 15 and Fig. 16, rests on the slope of the core so that only the mass of material in ΔABC provides resistance against shear in an upstream direction and the mass of material in ΔGHF provides resistance against shear from the core pressure in a downstream direction.

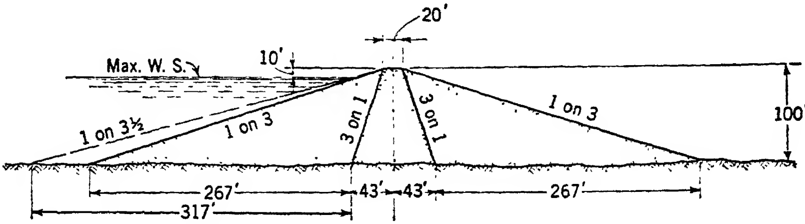


FIG. 16. Hydraulic fill dam analyzed (see Arts. 24 and 25).

ϕ_1 = angle of internal friction of core, = 18° (see Art. 23).
ϕ_2 = angle of internal friction of shell, = 35°, cot ϕ_2 = 1.428.

Shell material:

Specific gravity of shell material = 2.62
Per cent voids in shell material = 32

Dry weight per cu ft $= 62.5 \times 2.62 \times \left(\frac{100 - 32}{100}\right) =$ 111.3 lb. per cu ft

Saturated weight, add $0.32 \times$ weight of water $= 0.32 \times 62.5 =$ 20.0 lb per cu ft
Saturated unit weight = 131.3 lb per cu ft

Take moist weight of shell as 115 lb per cu ft and saturated weight as 131 lb per cu ft
Submerged weight of shell = 131 − 62.5 = 68.5 lb per cu ft

Core material:

Specific gravity = 2.62 per cent voids = 40% (average at completion)

Dry weight per cu ft $= 62.5 \times 2.62 \times 0.60 =$ 98.5 lb per cu ft
For saturated weight add 0.4×62.5 = 25.0 lb per cu ft
Saturated unit weight of core = 123.5 lb per cu ft

F_{su} = over-all factor of safety against horizontal shear in upstream direction,
F_{sd} = over-all factor of safety against horizontal shear in a downstream direction,
b_u = portion of base upstream from toe of core = base of ΔABC,
b_d = portion of base downstream from toe of core = base of ΔGFH,
h = height of dam above horizontal plane under consideration,
S_h = total horizontal shear in the upstream or downstream direction due to core pressure,
w_1 = unit weight of core material in actual wet or moist condition,
ϕ_1 = angle of internal friction of core,
W_u = total effective weight of the resisting ΔABC,
ϕ_2 = angle of internal friction of shell material,
α_u = upstream slope angle: upstream face with the horizontal,
α_d = downstream slope angle: downstream face with the horizontal,

W_d = total effective weight of the resisting ΔFGH,

$$S_h = \frac{w_1 h^2 \tan^2 (45° - \phi_1/2)}{2}.$$

$$F_{su} = \frac{2W_u \tan \phi_2}{w_1 h^2 \tan^2 (45° - \phi_1/2)} \qquad [22]$$

and

$$F_{sd} = \frac{2W_d \tan \phi_2}{w_1 h^2 \tan^2 (45° - \phi_1/2)} \qquad [23]$$

These two formulas give the over-all factor of safety against horizontal shear, which should be at least 2 with this approximate method.

25. Analysis of Hydraulic Fill Dam. *Example.* The hydraulic fill dam of Fig. 16 is similar to that of many actual dams, but it has been slightly idealized to the extent that both the upstream and downstream slopes are uniform throughout. The foundation is assumed to be of adequate shear strength.

Case 1: Safety just on completion of construction, which is frequently the most critical period in the life of a hydraulic fill dam. It will be noted that the downstream slope is the steepest and therefore it will be the critical one during construction.

The symbols in the Gilboy Eq. 20 have the following values from Fig. 16:

$$A = \tfrac{1}{3} = 0.333$$

$$B = 1.428 = \text{cotangent of angle of internal friction of shell material}$$

$$C = 3.0$$

$$C - A = 3 - 0.333 = 2.667$$

$$C - B = 3 - 1.428 = 1.572$$

$$\sqrt{1 + B^2} = \sqrt{1 + 1.428 \times 1.428} = 1.74$$

$$\sqrt{C - A} = 1.635$$

$$\sqrt{C - B} = 1.252$$

$$\sqrt{1 + A^2} = \sqrt{1 + 0.333 \times 0.333} = \sqrt{1 + 0.111} = 1.055$$

$$(1 + C^2) = 10$$

Substituting in Eq. 20,

$$\sqrt{R} = \frac{2.667(1.74) + 1.635 \times 1.252 \times 1.055}{10.0 - 2.667 \times 1.572}$$

$$\sqrt{R} = \frac{6.80}{5.80} = 1.17$$

$$R = 1.37$$

$R = 1.37$, which is the ratio of unit effective core weight to unit effective shell weight for a factor of safety of unity, assuming the core is a liquid.

Owing to the hydraulic process of construction, the shells just at the time of completion will be nearly 50 per cent saturated. Therefore, to get w, the ef-

fective unit weight of shell material in Art. 24, we will consider that one-half the shell material has moist weight equal to 115 lb per cu ft, from Fig. 16, and the other half has submerged (or buoyant weight) = 68.5 lb per cu ft because it is below the line of saturation. This gives the average effective weight of shell material $w = \dfrac{115 + 68.5}{2} = 91.8$ lb per cu ft.

The saturated unit weight of core material, from Fig. 16, is 123.5 lb per cu ft and $\phi_1 = 18°$. Applying Eq. 21

$$R_1 \text{ (of Art. 23)} = \frac{123.5 \tan^2 (45° - 9°)}{91.8}$$

$$\tan 36° = 0.727$$

$$\tan^2 36° = 0.528$$

$$R_1 = \frac{123.5 \times 0.528}{91.8} = 0.711$$

and the factor of safety against movement of the shells due to the pressure of the core is

$$F_s = \frac{R}{R_1} = \frac{1.37}{0.711} = 1.93$$

which is an ample factor of safety.

The above is the minimum over-all factor of safety during construction.

Case 2: Safety of upstream face against sudden drawdown. We will now investigate the safety of the upstream slope under conditions of sudden drawdown. It is assumed that the dam goes into service at once and that there is no opportunity for the further consolidation of the core before the sudden drawdown takes place.

In spite of the fact that the outer portion of most hydraulic fill dams is composed largely of relatively clean stone and coarse gravel, the rather far-fetched assumption will here be made that the shell cannot drain nearly as rapidly as the water surface can be drawn down and therefore the submerged unit weight of the material, 68.5 lb per cu ft, will be utilized as the effective unit weight of the shell material.

$$R_1 = \frac{123.5 \times 0.528}{68.5} = 0.954$$

$$F_s = \frac{1.37}{0.954} = 1.44$$

which is the factor of safety against sudden drawdown in the present case. The value of F_s is satisfactory when we consider the extremely severe assumption made. For almost all hydraulic fill dams the outer portion of the upstream shell would be so free draining that the full dry or moist unit weight of the material could properly be used for a large part of the upstream portion of the shell.

Case 3: Over-all factor of safety during construction against horizontal shear for dam of Fig. 16.

W_u (Art. 24), $\dfrac{89 \times 267}{2} = 11{,}880$ sq ft, average effective unit weight of shell as below = 91.8 lb per cu ft dry weight (assume shell 50 per cent below saturation line $\left(\dfrac{(115 + 68.5)}{2}\right)$.

$$W_u = 11{,}880 \times 91.8 = 1{,}090{,}000 \text{ lb per ft width of dam} = 545 \text{ tons}$$

$$w = 123.5 \quad \phi_2 = 35^\circ \quad \tan 35^\circ = 0.7 \quad \tan^2\left(45^\circ - \frac{18^\circ}{2}\right) = 0.548$$

Substituting in equation 22 gives

$$F_{su} = \frac{2 \times 545 \times 0.7}{\dfrac{123.5 \times 10{,}000 \times 0.548}{2000}} = \frac{764}{338.5} = 2.25$$

Note that the factor of safety for similar conditions by a method based on the Gilboy formula was found to be 1.93. This wide variation in results necessitates a factor of safety of at least 2 against horizontal shear. In this case because both the upstream and downstream slope are the same, F_{su} will also $= F_{sd}$.

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CHAPTER 19

DETAILS OF EARTH DAMS

1. Methods of Construction. The material for an earth dam is excavated from a borrow pit relatively close to the site, transported to the site, distributed, and compacted in its final resting place in the embankment.

The usual methods of construction are:

a. Construction in rolled layers,

b. Hydraulic fill method,

c. Semi-hydraulic fill method.

a. Under the rolled fill method the material is usually excavated by steam shovel, scrapers, or dragline, hauled onto the dam, deposited, spread, moistened, and rolled.

b. Under the hydraulic fill method, the material may be excavated by several convenient methods, but it is transported onto the dam and there deposited by the agency of water.

c. Under the semi-hydraulic fill method of construction, material is dumped near the upstream and downstream face of the dam to form rough levees. The space between these levees is then filled with water, and the material placed in or upon the levees is washed toward the center of the dam.

Earth dams are sometimes built without the use of any systematic method of compaction. In some cases the material is puddled, which usually means that the material is dropped into a pool of water maintained on the embankment. All such methods or variations of them should almost always be rejected as leading to dangerous construction.

The proper choice of the method of construction is dependent, in part, on the character of material available and in part on economic considerations. It should be the subject of careful investigation and study during the preliminary stages of the project. Quality, economy, and safety should all be given careful consideration.

2. Effect of Improvements in Equipment. In recent years there has been a marked increase in the size and efficiency of earth hauling equipment, and hauling units have been developed which will carry 20 cu yd or more at a time. If the haul is more than about 2000 ft these hauling units generally consist of a multi-tired tractor with a dumping trailer. If the haul is less than 2000 ft the big scraper pulled by a caterpillar tractor, such as the Le Tourneau, may be used as both an excavating and hauling unit.

Thirty years ago the capacity of the horse-drawn units was 1 or 2 cu yd each. In the case of the narrow gage construction railroad, the capacity per car was usually 4 to 6 cu yd.

An important thing to note is the tremendous increase in the capacity of units. This increase has meant a reduction in cost and a marked decrease in man-hours per cubic yard of material.

Similarly with the hydraulic fill method of construction, the size of equipment has greatly increased, but the change has not been as rapid as in automotive hauling equipment.

The net effect of the improvement and changes in earth hauling equipment has been that

a. There has been a relatively small decrease in over-all cost of earth dams. (That the decrease has not been greater is due to the coincident rise in the entire price structure.)

b. The range in which hydraulic fill dams might be used in competition with the rolled fill method has become somewhat narrower.

c. At present it is usually economically advisable to build small and moderate-sized dams by rolled fill methods.

d. In most cases it requires not only suitable materials close at hand but also particularly favorable conditions and a yardage in excess of 3 or 4 million cu yd to make it economically advisable to construct hydraulic fill dams. However, for very large earth dams the hydraulic fill dam continues to be frequently the most desirable and economical structure where suitable materials are available.

3. Importance of Careful Construction. An entirely safe and substantial design may be entirely ruined by careless and shoddy execution, and the failure

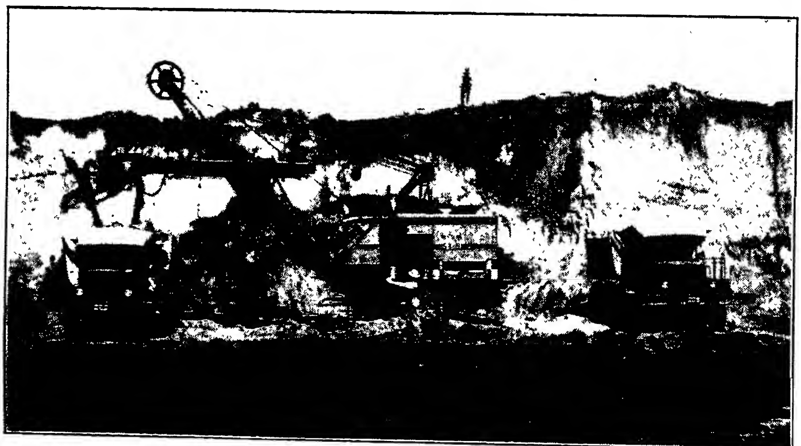

FIG. 1. Diesel operated power shovel 2½ cu yd capacity. (*Courtesy U. S. Engineer Office, Denison, Texas.*)

of the structure may very possibly be the result. Careful attention to the details of construction is therefore fully as important as the preliminary investigation and design.

4. Clearing and Stripping. About the first step that must be taken after the erection of the camp and plant buildings is the clearing and stripping of the site.

This work should, in fact, be done while the plant is being moved in and erected. Trees and stumps should be removed from the site. The requirements for stripping should vary with the existing conditions at the site and the requirements of the design. For instance, the original surface might be an unconsolidated fat clay which it might be advisable to remove because of its low shear resistance. As a general rule stripping involves only removing the sod (and not

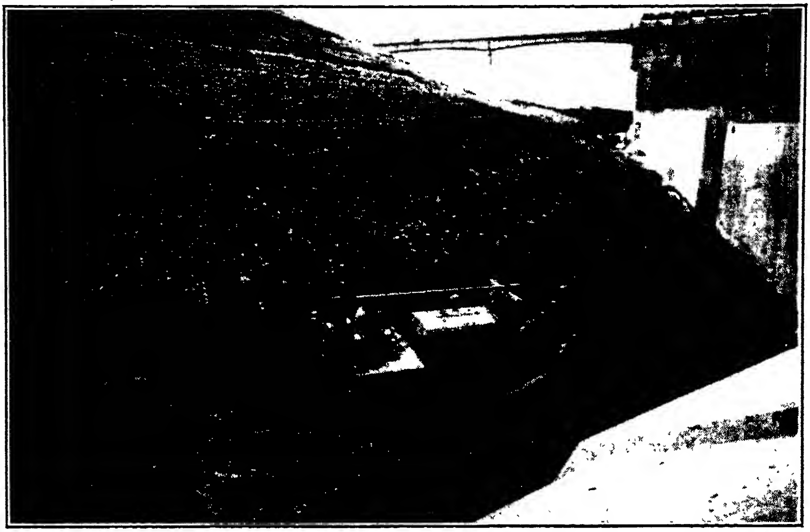

FIG. 2. Caterpillar R D 8 Bulldozer. (*Courtesy U. S. Engineer Office, Denison, Texas.*)

necessarily all the roots) and obtaining a bond between the embankment material and the foundation materials.

5. Removal of Vegetable Matter. Some specifications require that all soil containing more than 6 per cent of vegetable matter shall be removed from the site; other specifications limit the amount of vegetable matter permissible either in foundation or embankment to 3 per cent. The core of the North Dike of Wachusett Reservoir, which is built of soil containing about 6 per cent of vegetable matter, is giving entire satisfaction. It is the opinion of experts that it will take many hundreds and perhaps thousands of years for the vegetable matter to disappear from a core of that kind when saturated.[1] Even as much as 8 per cent of vegetable matter is sometimes permissible. In many cases the presence of a small percentage of vegetable matter helps to make the material more impermeable.

6. Bonding Dam to Foundation. It is a matter of prime importance to make sure that there is no definite dividing plane between the foundation material and the material composing the embankment. The original surface should be

[1] *Trans. Am. Soc. Civil Engrs.*, Vol. 48, 1902, p. 267.

plowed up and dragged with a disk harrow just before depositing any material on it. Unless the embankment is to be placed hydraulically, or the foundation surface already contains the optimum amount of moisture, the surface should be liberally moistened before the first layer is placed so that when the roller goes over this first layer it will force the new material down into the old and leave no dividing plane.

The mistake is sometimes made of starting the core trench before the site is cleared and properly stripped. No construction man likes to move dirt more than once, and there is a chance that some of the material removed from the core

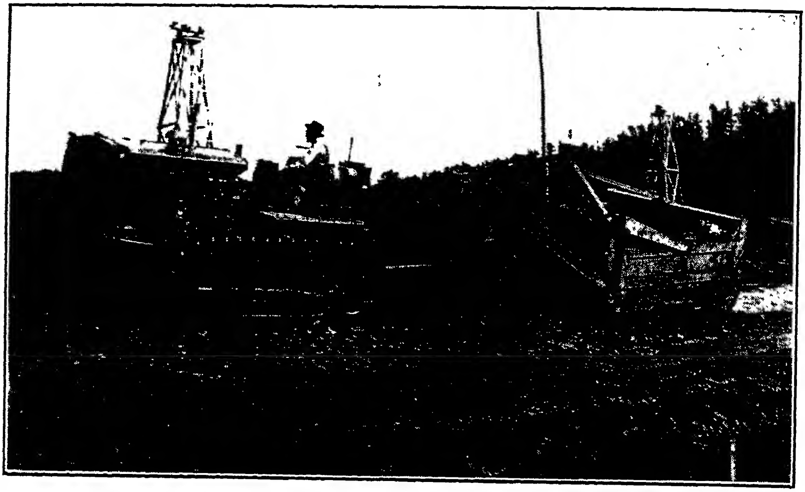

Fig. 3. La Tourneau Scraper 13 C.Y. Capacity powered by R D 8 Caterpillar Tractor. (*Courtesy U. S. Engineer Office, Little Rock, Ark.*)

trench will permanently remain covering sod, roots and stumps. Generally it is convenient to spread the material excavated from the core trench when it is excavated and roll it permanently in place. Then when it is necessary to refill the core wall trench this can be done with material from the borrow pits.

After stripping the site, no time should be lost in starting the core trench or sheet piling, for in such work difficulties are often encountered which delay the starting of the embankment work. It is well so to arrange the progress schedule that one may rely on the cutoff being completed to the level of the original ground surface by the time the main embankment is started. If this can be done it will greatly facilitate the embankment operations.

7. Puddling. The puddling of clayish soils in the cutoff trenches under dams is of sufficiently common occurrence to require some discussion here. This generally means that the trench is filled with water, and the dry material is then thrown in. It is a nefarious procedure to puddle any material which is high in clay. Clay will sometimes take up 2½ times its own weight of water, when it becomes a slimy mass exerting substantial hydrostatic pressure. Such clay

puddled in the core of a dam requires many years to attain a really stable condition. Also in drying it contracts and may leave cracks which produce roofing of the impervious overlying embankment section. A passageway through the impervious section is thus provided.

The drying of clay is particularly noticeable when a pond in a clay country dries up in the summer time. When the waters of the pond have evaporated, great cracks are seen in the surface of the clay, running in every direction. This shows how pure clay or material high in clay would react when puddled.

It is much better to unwater the trench and refill with compacted impervious

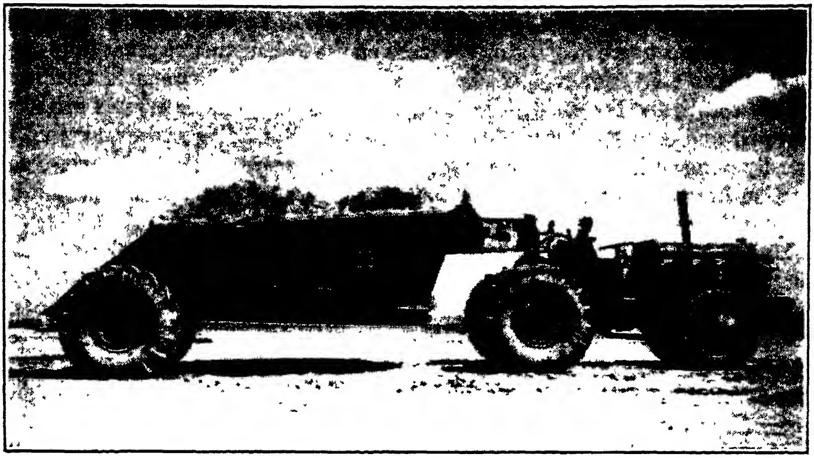

FIG. 4. Euclid six-wheel Diesel trac-truck, 13 cu yd capacity. (*Courtesy U. S. Engineer Office, Albuquerque, N. Mex.*)

material at a suitable moisture content. If puddling must be done use only silty sand for the purpose.

There is nothing better for a cutoff than a puddle trench refilled with genuine puddle. True puddle is an intimate mixture of stiff clay, sand, and gravel thoroughly tamped into place. Properly constructed, it is superior to concrete as a cutoff as the material itself is less pervious and also cracks through such puddle walls are practically unknown.

Cutoff trenches are frequently excavated to reach some layer which is more impervious than that immediately underlying the dam. Precautions should be taken to secure a bond between the material refilled into the trench and the material composing the layer which the cutoff trench reaches.

8. Building Embankment in Layers. Unless the material is to be placed and settled by the action of water, it should be spread in thin layers and rolled. Occasionally dams have been built by dumping the material from trestles much in the manner that railroad fills are made. Such a method should never be used, as it gives porous and unstable embankments which invite piping and sliding.

The layers should seldom be over 12 in. in thickness after rolling and usually very much thinner. Probably the best practice with the usual run of pervious materials is to require 8- to 10-in. layers, whereas cohesive materials that are readily compressible should be placed in layers 4 to 6 in. thick.

Having decided on the desired density, field experimentation on the thickness of layers, the amount of rolling, and the weight of the rollers is the only method of determining the economic manner of obtaining this density. These experiments should be coordinated with the control compaction tests with regard to moisture content. (See Art. 19, Chapter 16.)

9. Method of Depositing for Rolled Layers. With today's tremendous loads, 20 cu yd or more is often transported by modern hauling units, it is necessary to use great care in spotting the loads on the embankment so that they may be spread to a layer of even thickness and then rolled to the required thickness for the compacted layer. This is frequently 4 to 6 in. for impervious cohesive material and 8 to 10 in. for pervious materials. Spreading and evening up is usually done with a bulldozer and/or heavy road grader. To avoid over-compaction from hauling equipment, hauling routes on the embankment should be constantly varied.

10. Wetting Embankment. With impervious cohesive materials, the moisture content of material as placed should generally be as close to optimum moisture content[2] as practicable. Excessive "weaving" of the embankment under the moving load of rollers and trucks indicates to experienced engineers and inspectors that the material is too wet. In case moisture content is excessive the material should be allowed to lie until the excess moisture has evaporated or drained out before the layer is rolled. In some cases it may be advisable to adopt drainage measures in the borrow pit and sometimes it may be necessary to move to another borrow pit which does not contain an excess of moisture.

If tests indicate that additional moisture is desirable, it is well to add at least a part of this by sprinkling the layer after it has been thoroughly compacted and just before the additional material is added because then the pressure and kneading action of the roller will force the water through the material in a fairly even manner.

With very pervious materials it is usually almost impossible to add too much water and in order to get the best results from rolling, it is necessary to practically saturate the pervious layer immediately in advance of rolling.

11. Rolling Embankment. The sheep's-foot roller (see Fig. 5) is, in general, the best tool for rolling the embankment, although smooth rollers may be useful for following the sheep's-foot rollers to smooth up the embankment so that it will more readily shed the water in case of rain. The thickness of layers after rolling is generally 4 to 6 in. for impervious material and 8 in. or more for pervious material. The sheep's-foot roller may be arranged to give the desired unit pressure throughout a wide range up to 500 lb per sq in. (assuming one row of feet in contact). The variation in pressure is accomplished by adding or taking off feet and by loading the drum of the roller with water or sand or both. These

[2] For definition and method of determining see Arts. 2 and 19, Chapter 16.

high unit pressures obtainable with the sheep's-foot roller compare with approximately 50 lb per sq in. obtained with the old-fashioned steam roller with smooth wheel surface formerly used in compacting and rolling the successive layers in the construction of earth dams.

The unit weight of embankment desired is known in advance and will determine the desirable pressure to use on the sheep's-foot and also the number of passes of the roller. The amount of rolling to obtain the desired unit weight and the necessary moisture content is determined by experiment. Six to eight passes of the sheep's-foot are usual. In general the desired unit weight of the

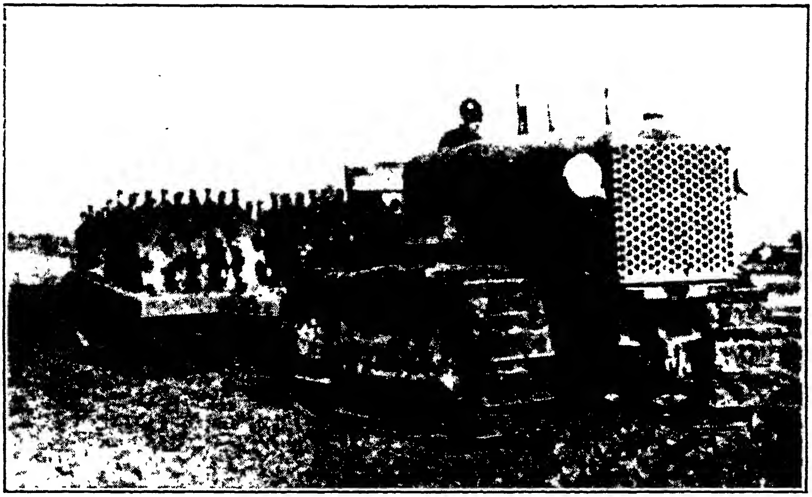

FIG. 5. Sheep's-foot rollers, three in tandem. Feet 7 inches high, 7 sq in. end area; approximate pressure 400 lb per sq in. Pulled by Allis-Chalmers D 8 Tractor. (*Courtesy U. S. Engineer Office, Little Rock, Ark.*)

embankment is somewhat greater than the unit weight which would be eventually obtained by natural consolidation due to the pressure of the portion of the dam above.

12. Danger of Over-Rolling. Equipment is now so heavy and unit pressures obtainable are so high that it is possible to over-compact the impervious embankment of an earth dam. In particular, the travel of the heavy hauling equipment over a cohesive embankment, such as one high in clay, should not be allowed to follow established ruts but should be spread out over a wide area.

Excess rolling or passage of heavy equipment on material with relatively high water content tends to cause local shear failure. Such a failure may be indicated by slickensides such as those shown in Fig. 6.

It is important to note that both moisture content and the compaction loading and its repetition are factors in this sort of construction failure. For instance, a heavily loaded sheep's-foot roller making a certain number of passes over an embankment layer high in clay may produce shear failure in the material as evi-

denced by the presence of slickensides. However, if the moisture content of this material is sufficiently reduced the same roller equipment with the same number of passes will not produce shear failure in the material.

In order to avoid over-rolling of cohesive material with possible shear failure, one should

1. Keep the material slightly on the dry side of optimum moisture content (see Art. 19, Chapter 16). It should definitely be on the dry side of Atterberg's plastic limit. As water is usually added on the bank this is not so difficult.

FIG. 6. Slickensides due to over-compaction.

2. Be content with the predetermined desired dry unit weight of the materials. Make frequent tests to determine dry unit weight which you are getting (see Art. 19, this chapter), and do not roll beyond the point which will give the desired unit weight.

3. Watch the action of the roller on the embankment. Excessive weaving of the embankment under equipment is generally indicative of excess moisture and possible trouble.

4. Require the heavy hauling equipment to follow diverse routes across the impervious section and avoid continuing tracking in the ruts formed by prior passage of equipment.

At Tappan Dam, Ohio, during construction, on August 26, 1935, there was an upstream movement amounting to a maximum of 7½ ft producing horizontal

shear in the foundation and opening vertical cracks in the upstream face. This accident, it is believed, was in part due to over-rolling and in part due to excess moisture in the embankment and foundation. In fact, these two causes are to a certain extent complementary. The foundation at Tappan was clay about 11 ft thick underlain by sand and gravel. The embankment material with a high clay content was placed on the wet side of optimum moisture content. Much of it was at or slightly beyond the Atterberg plastic limit. The design height of the dam was about 60 ft, but when the embankment reached a height between 20 and 30 ft a number of vertical longitudinal cracks appeared in the upstream face all the way to the top of the embankment. Shear planes and slickensides were located in the embankment and near the top surface of the clay foundation. Corrective measures, including a berm, were adopted and the dam was successfully completed.

13. Compaction of Pervious Material. Unlike impervious cohesive materials, there is no danger of over-rolling or over-compacting pervious cohesionless materials such as sands and gravels. While sands and gravels reach their ultimate consolidation under any given loading almost immediately, there are benefits to be derived from a thorough compaction which is greater than the natural consolidation resulting from the weight of the embankment. In the case of uniform materials such as blow sand which in nature may be even less dense than critical density (see Art. 13, Chapter 16), a sufficient amount of compaction will consolidate the material to one which is far on the safe side of critical density and will also materially increase its shear strength, thus permitting economy in the use of materials. Even for well-graded cohesionless materials which are well on the safe side of critical density, some compaction is desirable for improvement of shear strength and the limitation of embankment settlement.

Unfortunately, many construction men still believe that it is impossible to compact sands and gravels. This probably arises from the fact that if the material is dry or merely moist, one can accomplish very little by rolling. The belief is definitely ill-founded, but in order to get substantial compaction it is necessary to practically saturate the material just before rolling. In fairly clean coarse gravel the benefits obtained are generally not worth the extra cost. With a well-graded sand gravel the additional cost is not great, and usually wetting and rolling should be done. With fine uniform sands thorough wetting and rolling is absolutely essential.

14. Slope of Layers. The layers of a rolled fill dam should be deposited in an approximately horizontal position. The layers should be sloped away from the center of the dam. This slope should be from 1 on 20 to 1 on 50, enough to shed water readily. With materials containing a considerable percentage of clay, embankments sloped thus and rolled with a plane roller before the rain will shed rains quickly so that the embankment may be worked on the very next day after the rain has stopped. On the other hand, embankments which are allowed to slope toward the center line get so wet and mired up from a little rain that the pool thus formed has to be ditched and drained away and several days are lost before embankment operations can be resumed. Certainly no construction man

who has tried dipping the embankment toward the center in a country where there is much rain will try it a second time.

15. Precautions Around Structures. It is often difficult to roll up close to core walls, conduits, abutment walls, etc. All structures in or against an impervious embankment which serves as a water barrier should have a batter on the side in contact with the earth so that settlement will produce a tighter contact with the earth. Baffles also with a batter should be used as an additional safeguard.

Special pains must be taken to see that the material along these structures is properly compacted.

Places where the roller cannot reach should be tamped in thin layers with compressed air tampers to a density equal to that obtained by rolling. Special care should be taken in tamping the embankment along the face of structures which pass through the dam in an upstream and downstream direction. Material which is deposited against the face of such structures should be spread and thoroughly tamped with compressed air tampers in 3-in. layers. Each such layer should be sprinkled to the same extent as the embankment which is to be rolled.

16. Settlement of Embankments. Settlement depends on the character of the material in the embankment and the foundation and the methods of construction used. It is customary to construct earth dams to a somewhat greater height and width than the neat dimensions called for by the plans. For an embankment rolled in 6-in. layers of impervious and pervious materials, there is no reason for anticipating any appreciable settlement in the embankment itself, but settlement may occur in the foundation. For a rolled fill dam on an unyielding foundation a nominal allowance for settlement of 1 per cent is sufficient.

For hydraulic fill dams, settlement due to consolidation of the core must be anticipated and will vary with the character of the material. With modern methods (narrow cores and wastage of the excessively fine fines, including colloids), the allowance for settlement of the embankment (after completion) need never exceed 4 per cent. This is exclusive of foundation settlement.

Foundation settlements have ranged all the way from practically zero up to 8 per cent of the height of the dam or more, most of which takes place during construction. Deep plastic foundations of relatively unconsolidated clays show the greatest settlement.

Considering settlement of both foundation and embankment, the total provision which should be made for settlement after the completion of the dam will range from a nominal 1 per cent of the height of the dam to a maximum of 6 per cent or more. Thus if 6 per cent had been determined as the proper allowance for settlement after completion and the dam was 100 ft high, it would actually be constructed to an elevation of 106 ft above the original foundation elevation.

It is possible to predict the amount of settlement to be anticipated with a fair degree of precision by utilizing the tests and methods outlined in Art. 18 of Chapter 16. This should be done and after deducting the computed settlement (or consolidation) during construction in both foundation and embankment the

remainder should be added to the designed elevation of the dam to obtain the elevation to which it is to be constructed.

17. Selection of Material During Construction. It is assumed that before the construction is started, the borrow pits have been thoroughly prospected and that it has been definitely decided in advance where the material for the upstream and downstream portions of the embankment is to come from. However, during construction, there are often variations in the material which were not indicated by the test pits and boreholes. Studies based on tests made of the borrow materials as actually obtained may result in some changes in design in the interests of economy or safety or both. Contractural relations, however, should be carefully considered before making changes unless such changes are essential to safety. Otherwise an intended economy may actually turn out to be a substantial additional cost.

18. Trimming of Slopes. Sometimes the slopes of earth dams are left without being trimmed to the neat lines of the upstream and downstream face as shown on the plans because it is felt by some that so long as there is the required amount of material in place, it is simply a waste of money to trim down the slopes. Contractors sometimes make statements similar to this: "It is true that the slope near the top is pretty near 1 on 1½ but you know that I've got in more material than the neat lines require and I don't see any sense in going to the expense of pulling down all that material to a 1 on 3 slope just because the specifications say so." It is further argued that the action of the water or the weather will soon wear down the slope to approximately the specified incline.

If any such ideas are allowed to influence the construction, accidents will be more than likely to occur. To leave a mass of material on either face at an incline steeper than that at which it will safely stand after the dam is placed in service is a very dangerous procedure as it leaves a superimposed load on the face of the dam. Such a load may remain stable for a time before the water is raised in the reservoir. Assume, for instance, that the upstream slope of an earth dam is 1 on 3 but that the top 20 ft has been left at a slope of 1 on 1. This material on a steep slope at the top of the upstream face forms a superimposed load. When the water in the reservoir is raised, the upstream face becomes saturated. Then say the reservoir level is lowered, and under the pressure of this superimposed load a slide is apt to occur. On the downstream face, when the line of saturation cuts the base near the toe, a similar loading is very apt to induce a slide. The folly of permitting such construction may also be demonstrated mathematically in any particular case by the methods of stability analysis outlined in Chapter 18. The hazard here indicated should always be avoided by keeping all slopes trimmed substantially to the designed lines.

19. Engineering Control of Rolled Fill Dam Construction. Before construction is started, the field and laboratory investigations of the borrow pit material have been made as discussed in Chapters 1 and 16. As a result the dry weight per cubic foot or dry density which it is desirable to have in the dam has been determined. It is the business of the engineering construction organization to obtain the desired density as nearly as practicable.

For a given density or dry weight per cubic foot of cohesive materials there is a certain amount of moisture which the material should have that is the optimum moisture content (see Art. 19, Chapter 16).

By a little experimenting the construction engineer determines how much water load he should add to his sheep's-foot rollers and how many passes of the rollers he must make over a 6-in. layer of the material in order to obtain the required dry weight per cubic foot in the embankment. He also determines how much water he must use in order to approach the optimum moisture content. As mentioned elsewhere, the engineer will, with most materials, find it advisable to keep the moisture content a little on the dry side of the theoretical optimum as determined by the Proctor tests (see Art. 19, Chapter 16).

In order that the engineer should know the dry density of the material which he is getting, it is necessary to constantly make determinations as the embankment work progresses. One good way for determining both wet and dry density in the field which is in quite common use is as follows:

Dig a small hole, say, 8 in. square and 8 in. deep in the surface of the embankment after first having cut the surface where it is being disturbed by the sheep's-foot roller down to firm hard material. Save every bit of material removed from the hole and weigh it as soon as it is taken out of the hole. Then fill the hole with fine dried sand, using a cylindrical graduate to measure the exact quantity used. Thus having both volume and weight, the unit moist weight of the embankment is readily found.

To get the moisture content or per cent moisture, the material is taken to a nearby shanty and a small sample which has been protected as much as practicable from the atmosphere is pan-dried and weighed both before and after drying. The moisture percentage may then be computed, and from this the unit dry weight of the embankment sample is obtained.

For any given embankment job a standard procedure should be set up so that all field tests for determining density will be the same. Two men can usually take care of this field test work on a job where 10,000 cu yd per day is placed and keep inspectors and engineers informed of the dry density and per cent of moisture for the material being placed.

The Proctor needle as described and illustrated in Art. 19, Chapter 16, is in many cases a useful tool in the control of embankment work.

Once the construction engineer and inspector become familiar with the particular requirements and materials in any given case, they will be able to obtain the desired results without an excessive number of tests for dry density. In addition to the determination of the optimum moisture content already mentioned, there are several less precise methods of determining the amount of water to add to a cohesive material before rolling.

One old-fashioned test for the proper amount of moisture in a cohesive embankment material is to take a small sample of the material between the hands and roll it out. If one could roll it out to just the diameter of a pencil the material was slightly wetter than desirable. If in rolling it became crumbly, then it was too dry and additional water should be added on the bank. If the roll was

just slightly crumbly, then the material had just about the proper moisture content for successful rolling.[3] This is practically the same thing as saying that the moisture content of cohesive materials before rolling should be slightly less than that required for the Atterberg plastic limit (see Art. 2, Chapter 16).

Another rough guide to proper moisture content is the action of the roller on the embankment. If the roller weaves up and down to an excessive extent it is certain that there is too much moisture in the embankment. If the embankment weaves only slightly on rolling, the moisture content is probably not far from that which is desirable. Of course, if the moisture content is very much in excess of the optimum, it is utterly impracticable to properly compact the embankment and the result may be an unstable embankment. With pervious embankment materials it is also necessary to determine the dry density, but there is never any danger of getting materials of this sort too dense.

20. Protection of Upstream Slope. It is always necessary to protect the upstream face of a dam from wave action to some extent. Protection should be provided for that portion of the upstream face which may be subjected to wave action.

In relatively small reservoirs log booms placed in front of the slope have proved to be effective in breaking the waves before they strike the slopes. For many

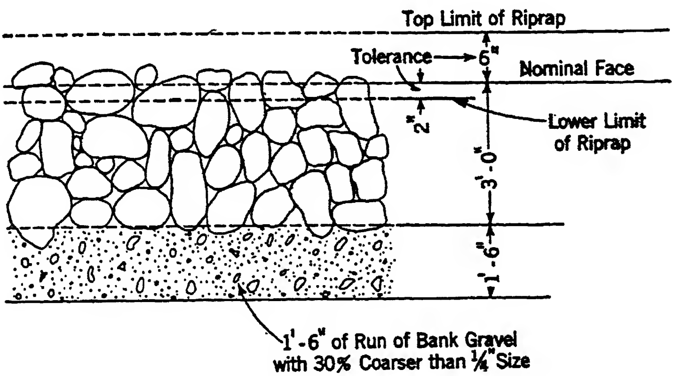

Fig. 7. Dumped riprap.

years the Spring Valley Water Co. of San Francisco, Calif., protected the earth slopes of its reservoirs by anchoring booms about 3 ft from the upstream slope.

In reservoirs that are emptied frequently, the bank protection should begin at the upstream toe. If it can be determined that the reservoir will not be drawn down below a certain elevation, the bank protection need not go below that elevation, but at that point a berm should be provided.

21. Riprap. There is no material superior to good stone riprap for the protection of the upstream face of an earth dam from wave action. Stone riprap is of two classes, random riprap and hand-placed. The former consists of stones

[3] As an indication as to whether or not additional water should be added before rolling this test has been used at least since 1910 and probably much longer.

dumped in place from cars or trucks or tossed into place by hand. If it is dumped in place from cars or trucks, the individual stones may be any size up to the capacity of the steam shovel, such material sometimes being termed cyclopean riprap or derrick stone riprap. All riprap should be placed on a bed of gravel, and derrick stone riprap should have 18 in. of spalls on top of the gravel. Dumped riprap should not be less than 3 ft thick and generally 3 ft of dumped riprap is considered the equivalent of 18 in. of hand-placed.

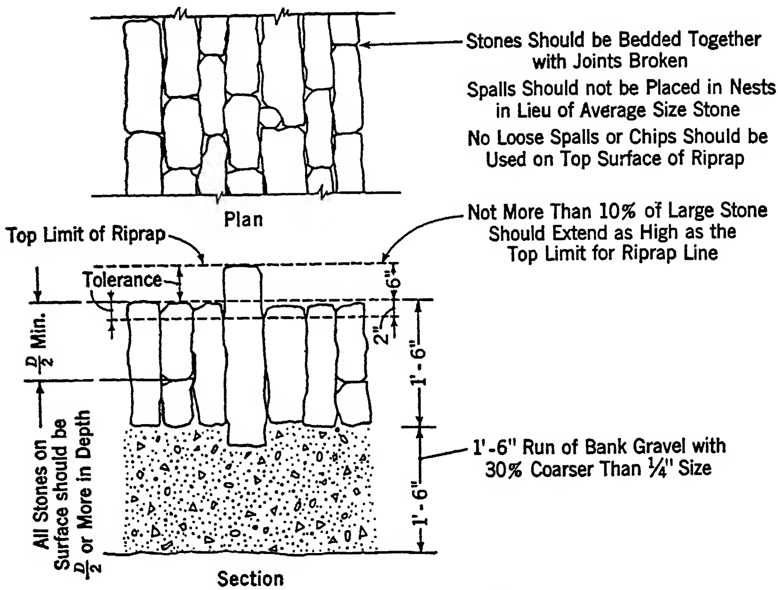

FIG. 8. Hand-placed riprap.

Hand-placed riprap consists of one-man stones laid on edge on a gravel bed that has been prepared and graded. An effort is made to break joints as much as possible, and the voids are filled with smaller stones. Hand-placed riprap is generally 18 in. thick, the minimum size of individual stones frequently being specified as 12 by 12 by 3 in. It is also often required that at least 50 per cent of the surface shall be of stones which are 18 in. deep.

The best hand-placed riprap approaches good dry rubble in quality and appearance. The bottom band of riprap on the upstream face of an earth dam should rest on a shoulder or berm in the embankment; otherwise, the weight of the riprap might be sufficient to cause it to slide down the saturated upstream face. The bottom course should be formed with headers twice as deep as the other stones and set into the bank in a trench at the inner edge of the berm.

All riprap should be laid on a bed of gravel or crushed stone grading in particle size from coarse sand up. In order to prevent waves from washing out the underlying material through the voids in the riprap and destroying its support, all

riprap should be laid in a bed of gravel or crushed stone or spalls which grade from a size that will prevent the material of the dam from washing into it up to a size that will not pass through the voids in the riprap. The thickness of the above gravel or crushed stone layer underlying the riprap should vary according to its character from 9 to 18 in.

On the other hand if the material in the upstream portion of the dam was a clayish silt and the bedding layer for the riprap was a run of bank gravel containing only 30 per cent larger than ¼ in. but grading up to a coarse gravel, it might be advisable to make this layer as much as 18 in. thick. In general the thickness of the bedding layer should never be less than 9 in. and then only when properly graded. Action of the waves will wash out some of the finer bedding material and, with some settlement, will automatically build up a stable filter layer, varying from fine material at the bottom to coarse material at the top. Settlement will be greater for run of bank gravel than for artificially graded gravel, which is to be preferred. Therefore the run of bank gravel should be thicker.

Under no circumstances should the bedding layer be omitted unless the material in the upstream face is gravel. If it is omitted for a clay or silt upstream embankment, the action of the waves will in time wash the material away from between the riprap stones and will move them and even sometimes bury them in the embankment material.

Not all rock is suitable for use as riprap. Stone for riprap should be hard and durable and should not break down readily on long exposure to water, frost, and air. Most but not all of the igneous and metamorphic rock makes good riprap. Many of the limestones and sandstones make excellent riprap.

Shales are generally entirely unsuitable and the presence of shale seams in limestone and sandstone sometimes renders these rocks undesirable. As a practical matter we usually take the best rock available at reasonable cost, but certainly it would not be worth while to consider any rock for riprap unless it could successfully withstand more than five cycles of freezing and thawing (see Art. 7, Chapter 16).

In investigating the available rock for riprap, the available records of service of the rock in the territory should be investigated as of at least equal importance with the freezing and thawing test. In addition to its record of use in riprap, its record in bridge piers, in bridges, and in buildings is of interest.

22. Concrete Lining of Upstream Slopes. Municipal distribution reservoirs are often lined with concrete, which is generally laid in the form of blocks on graded slopes. Such linings are generally not relied on for watertightness but are intended primarily to furnish a surface on which wave action will not muddy the water. Such a surface may also be cleaned when the reservoir is drawn down.

Monolithic Concrete Lining. A concrete lining is sometimes used on the upstream face of earth dams. It is generally poor practice to place much reliance on a concrete pavement for keeping water from entering the embankment. The true function of such a lining, it is believed, is wave protection only. Sometimes the concrete lining is built as a reinforced monolith, steel composing about 0.3 per

cent of the effective area of the cross-section of concrete. In most cases such monolithic linings are of questionable value. When the bank settles, the reinforced concrete lining will be left unsupported at some points, and there the slab will crack and may eventually be broken up by wave action, thus allowing the waves to enter the gaps with disastrous effect.

Concrete Lining of Square Blocks. In some cases the upstream face of earth dams has been protected from wave action by concrete linings of square blocks, generally not larger than 6 by 6 ft. It is not usually necessary to reinforce these blocks. The thickness of the block in inches should be the same as the dimension of the block, in feet; i.e., a block 6 ft square should be 6 in. thick. The squares should be poured alternately, the blocks being separated by layers of three-ply tar paper so that they will adjust themselves to the surface of the embankment in case of settlement. Sometimes precast concrete slabs of much smaller size are used. Small blocks of wood, ½ in. thick, may be used to separate the slabs, thus insuring their settlement with the embankment and preventing bridging. Parallel to the center line of the embankment a concrete curb should be built against which to place the concrete lining. This curb should be at the inner edge of a berm or shoulder in the embankment and should extend not less than 18 in. below the bottom of the concrete lining.

If the concrete paving is monolithic or if the concrete blocks are set without spaces between them, it is essential to provide numerous gravel backed weep holes through the concrete (equal to at least 15 per cent of total area) in order to allow the water in the embankment to drain away when the reservoir is drawn down quickly. Otherwise, hydrostatic pressure behind the concrete lining might break it or cause it to slide down the slope and possibly, also, cause the sloughing of some of the saturated embankment material.

The accident at Belle Fourche Dam (see Art. 6, Chapter 17) illustrates the hazard to which concrete slab paving for wave protection is subject. It is believed that such slab paving should not be used unless adequate provision is made for back drainage.

The subject of wave protection is worthy of more thorough investigation than it has so far received. Its importance is indicated by the fact that it often costs from 25 to 50 per cent as much as the embankment itself.

23. Porous Concrete Slab Slope Protection. Porous concrete has been used for drainage purposes in masonry dams for at least 35 yr. When properly made it is comparatively strong and is very nearly as pervious as ordinary gravel or crushed stone. At the Santee-Cooper Project in South Carolina (1940), L. F. Harza, Consulting Engineer, decided to utilize a layer of porous concrete slab for the protection of the upstream slope of some of the earth dams. The climate is mild and the formation of anything but paper-thin ice on a pond or reservoir is unheard of.

Porous concrete for slope protection has the same advantage as stone riprap, i.e., that on the recession of a wave the water can readily drain out through the interstices without the accumulation of a considerable amount of hydrostatic pressure against the bottom of the slab.

The porous concrete was laid on a 4-in. bed of ⅛ in. to ½ in. cleaned gravel as indicated in Fig. 9. Fig. 10 shows the construction of the porous slabs.

At Santee Cooper about 1 bbl of cement was used per cu yd of porous concrete and the mortar was Portland cement paste without any sand. The aggregate

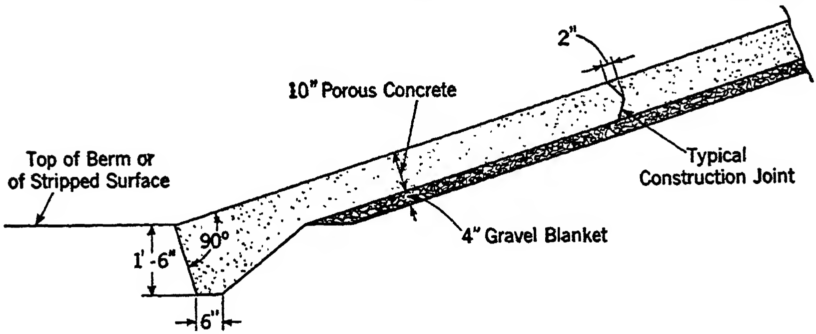

FIG. 9. Porous concrete slab slope protection in lieu of riprap as used by Harza Engineering Co. on Santee Cooper project, South Carolina.

FIG. 10. Placing porous concrete slab, Santee Cooper project. (*Courtesy Harza Engineering Co.*)

for the pervious concrete graded from ¾ in. to ¼ in. with about 50 per cent coarser than ⅜ in. This aggregate was thoroughly mixed with Portland cement with only enough water to provide a stiff cement paste which would adhere to the gravel and which would not tend to settle to the bottom of the porous concrete layer and fill the voids.

The use of porous concrete slab slope protection appears to have further application in a location where stone riprap is difficult to obtain or is very expensive. Its use should be confined to mild climates where ice does not form on reservoirs.

24. Berms. In earth dams more than 30 ft high berms from 6 to 20 ft wide should be used on the downstream face. In high dams berms should be used for about each 30-ft difference in elevation. The main function of berms is to minimize the erosion from rainstorms, the effects of which may be very severe. The outer edge of berms should be higher than the inner edge, in order to prevent rain water from flowing over the edge and down the slope.

A gutter should be placed at the inner edge of the berm and given a slight grade to conduct the storm water to the side of the valley, where other gutters or storm drains conduct it to the toe of the dam. In many of the largest and highest earth dams, the storm water from the berms is collected by catch basins and conducted through storm sewers to the main drainage system at the downstream toe of the dam.

On some dams, berms are built on the upstream face, and a berm should always be used as a shoulder against which to build the bottom of the riprap.

25. Protection of Top and Downstream Face of Dam. There is no better protection for the downstream face of an earth dam than stone riprap on gravel. If rock is plentiful and close at hand, such protection is often more economical than the use of a grass cover and top soil with the necessary drainage features as discussed in Art. 24 above, together with the continuing expense of maintenance.

Where stone protection is not practicable, protection from erosion may be obtained by building up a suitable growth of grass or vines. All the exposed parts of the dam should be covered with 12 to 18 in. of rich top soil. The heavier layer of top soil has been found necessary where the material under it is very pervious. The top soil should be treated with about 600 lb of good fertilizer per acre. The surface should then be freshly raked and seeded to grass. A good seed mixture for this purpose in a temperate and moist climate is red top, 12 lb; white clover, 6 lb; and Canadian blue grass, 10 lb of the recleaned seed per acre. In the South, Bermuda grass and "wire grass" have been used with great success.

Matrimony vine (*Lycium vulgare*) has been successfully used for bank protection. The shoots of this vine are planted, and the vine grows along the ground, sending out roots which enter the ground for some depth and then start other shoots. The vine, which was brought from the Mediterranean, is very hardy. In a few seasons the bank presents the appearance of a continuous tangle of vines, and the top soil becomes a tangle of roots.

26. Talla Dam. The Talla Dam of the Edinburgh, Scotland, waterworks, which was completed in 1906, may be considered more or less typical of many successful earth dams both abroad and in America constructed in the same period except that in America a concrete core wall was usually substituted for the puddle wall.

The typical cross-section of the Talla Dam is shown in Fig. 11. The maximum height of the dam is 80 ft and the length 1050 ft. The structure contains about

500,000 cu yd of material. The freeboard is 7 ft and the top width 20 ft. The upstream slope is 1 on 4 and the downstream 1 on 3 with berms near the top and bottom. The core of the dam consists of a heavy clay puddle sealed into the rock. The maximum depth of this clay puddle wall below the original surface of the ground is 82 ft. At the top of the embankment the width of this puddle wall is 10 ft, whereas at the surface of the original ground its width is 32 ft. On each side of the puddled core in the central portion of the dam the clay material

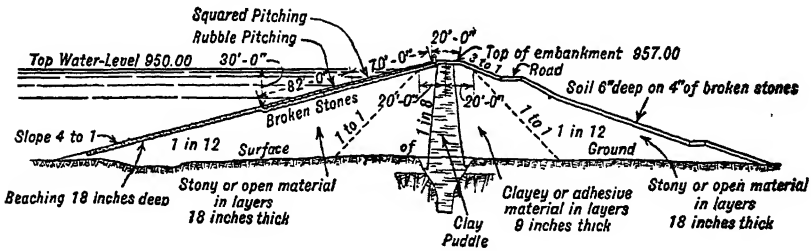

Fig. 11. Talla Dam. (*From Eng. Record, Vol. 57, p. 21.*)

was rolled in 9-in. layers. The outer portions of the embankment are composed of stony or open material placed in 18-in. layers. From the top of the embankment to a point 10 ft below high water level there is a face protection of 12-in. squared blocks of stone set in 12 in. of broken stone. Below this is a facing of 12 in. of rubble to a point 30 ft below the high water mark. The remainder of the slope is covered with 18 in. of gravel.

"Puddle wall" as here used means an intimate mixture of stiff clay, sand, and gravel tamped into place. As a barrier to the percolation of water, it is believed to be superior to a concrete core wall.

For a fuller description of Talla Dam, see *Earth Dam Projects*, page 233.

27. North Embankment of John Martin Dam. The John Martin Dam on the Arkansas River, near Caddoa, Colo. (1942), is a flood-control and irrigation project. The central section of the dam is a gravity overflow concrete masonry section equipped with tainter gates. On the north and south wings, earth dam sections 3700 and 5800 ft long are utilized. The north wing dam has an approximate height of 150 ft throughout the valley section.

The width of the valley bottom between ledge rock abutments at the site is about 4200 ft, of which 1640 ft is taken up by the masonry section. Over this width of 4/5 of a mile, the Arkansas River has wandered back and forth during recent geologic times. A flood plane has been built up, the top 5 to 10 ft of which consists, in general, of quite impervious silt. Below this top layer the underground consists of coarse sand gravel and boulders, 25 to 40 ft deep to ledge rock, which is a firm hard sandstone.

When the exploratory work revealed the presence of large boulders in the extremely pervious underground, it was feared that it would not be practicable to drive steel sheet piling. Accordingly, a shaft lined with steel sheet piling was driven to ledge rock and excavated without any particular trouble. Steel sheet

piling was accordingly utilized to obtain a cutoff through the highly pervious underground of coarse sand gravel and boulders. Carnegie Section M 112 with ⅜ in. thick web, 23 lb per sq ft was used except where an old river channel increased the depth from about 30 ft to about 50 ft, where M 113 with a web thickness of ½ in. (28 lb per sq ft) was used. To promote watertightness and lubrication, interlocks were swabbed with Philip Carey "Noah's Pitch" mixed with long asbestos fiber. Practically no trouble was experienced in driving and seating the steel sheet piling in the sandstone. Fig. 12 shows some of the sheet piling cutoff being driven.

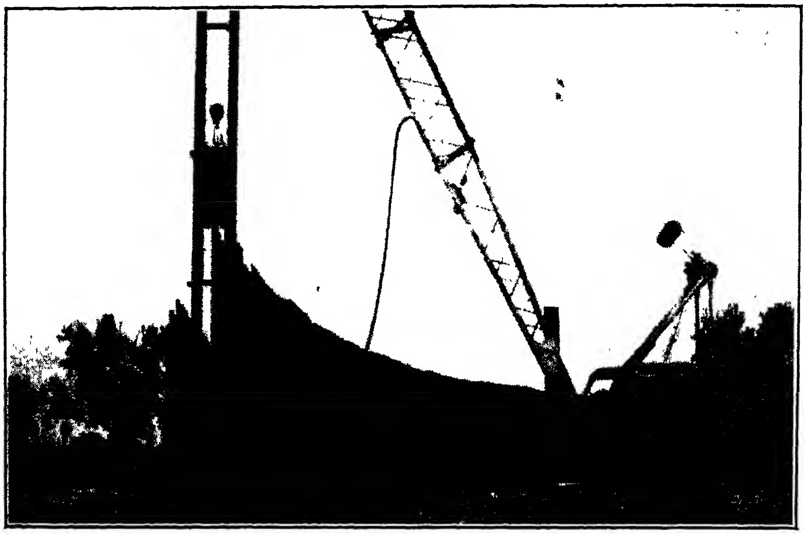

FIG. 12. Driving sheet piling, cutoff north embankment John Martin Dam. (*Courtesy U. S. Engineer Office, Caddoa, Colo.*)

In Fig. 13 it will be noted that the character of the material in the foundation and from the borrow pits permitted relatively steep slopes both upstream and downstream. It will be noted that the steel sheet pile cutoff is located upstream from the upstream toe of the dam and is joined to the impervious section of the dam by means of an impervious blanket, which extends from the impervious central section of the dam upstream for a distance of 1500 ft from the steel sheet piling cutoff. Except under the pervious upstream section of the dam, there was very little work to be done in order to establish the impervious upstream blanket shown in Fig. 13. This is because over most of the area there is already a satisfactory impervious blanket in place. For the most part this work consisted in removing vegetation, then patching and rolling. In this case the blanket is simply an extra precaution available at very slight extra cost.

Downstream from the steel sheet pile cutoff, the highly pervious underground acts as a very effective drain. Consequently, seepage pressure tends to be

downward. As a result, forces due to possible sudden drawdown are mitigated to a considerable extent (see Art. 7, Chapter 18). As a result, for the same factor of safety against sudden drawdown, it is practicable to have the slope of the upstream face steeper than would be the case if the cutoff had been located in its conventional position at the center line of the impervious section.

The apparent saving in this upstream position of the cutoff is not by any means a net saving. On the other side of the equation one has to consider the cost of the impervious horizontal blanket to join the upstream cutoff to the impervious section of the dam at the center of the dam and in some cases other items of cost such as the curving around of the cutoff to tie into the masonry

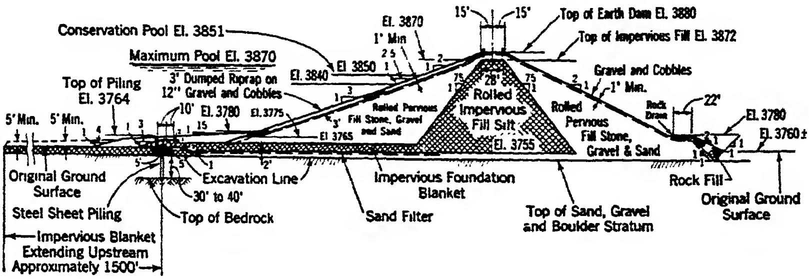

FIG. 13. North embankment, John Martin Dam, Caddoa, Colo.

section (with an increase in total length) and also the additional cost of bonding the horizontal blanket to the ledge rock abutment.

It is also possible that the impervious blanket under the upstream pervious shell may be enough weaker in shear than the pervious shell to require just as flat an upstream slope as would be required if the cutoff were at the center line.

All in all, except under special conditions, it is believed that from an economic standpoint, it is just as well to continue the usual practice of locating the cutoff, when used, under the central impervious section of the dam.

At the John Martin Dam, an ample quantity of material was available for both pervious and impervious portions of the embankment. The impervious material contained very little real clay and as a result both the pervious and the impervious sections showed a high shear strength on test. Owing to the arid climate, it was necessary to prewet some of the borrow pits for impervious material in some cases as much as 2 months in advance of starting excavation.

28. Conchas Dikes. The Conchas Dikes (1939) were constructed in connection with the Conchas Flood Control and Irrigation Project on the South Canadian River, about 60 miles from Tucumcari, N. Mex. The project included a concrete overflow dam 235 ft high.

The design of the section as shown by Fig. 14 was, as usual, materially influenced by the degree of availability of the various materials utilized. Thus, there was plenty of excellent impervious material readily available; 15 per cent of it, on the average, being of clay sizes and the remainder grading nicely up to a coarse sand. The pervious material of sand and gravel, on the other hand, was

somewhat more limited. In fact, in the course of the work, it was found that the pervious material was somewhat more limited than was anticipated. Because of the high shear strength of both pervious and impervious materials this did not cause much trouble, as the dividing line between pervious and impervious could be varied without materially affecting the strength of the section. It is this revised section which is shown in Fig. 14, and the dam was constructed substantially as there indicated. There was plenty of rock available nearby and hence, as shown in Fig. 14, a rock section was used for both upstream and downstream face.

To have both slopes protected by rock is a material advantage in this arid section as the production and maintenance of a stand of vegetation to protect the slopes is difficult, expensive, and frequently unsuccessful.

The maximum height of the Conchas Dikes is approximately 100 ft and the total length is over 2 miles. Ledge rock is, in general, from 5 to 10 ft below the original ground surface and is either a dense firm sandy shale or a good quality

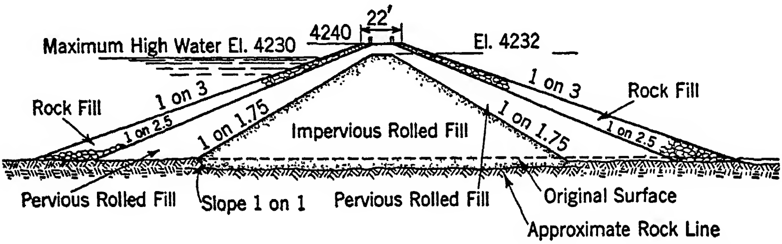

FIG. 14. North and South Dikes, Conchas Dam, New Mexico. (*From "Design and Construction of Conchas Dam," U. S. Engineer Office, Caddoa, Colo.*

of sandstone. Only loose rock was excavated, and as soon as solid ledge was encountered, it was utilized as the foundation of the dike. In many cases there was no rock excavation.

When the shale surface was cleaned up, preparatory to starting the impervious section of the embankment, it was found to present quite a smooth appearance. This led to the question as to whether water would seep along the relatively smooth surface of the rock and also as to whether the frictional resistance presented would be sufficient to make the section safe against sliding on the rock. In order to see whether or not a cutoff trench was required, a series of tests were made which showed that

a. The contact between the rolled impervious fill and the relatively smooth ledge rock was so intimate that water would not seep along the rock surface.

b. Compacted impervious embankment placed on the relatively smooth ledge rock could not be made to slide. When the horizontal pressure was increased sufficiently, the compacted embankment material sheared through and the full shear strength of the material was developed.[4]

[4] See "Design and Construction of Conchas Dam," Vol. 1, p. 144, U. S. Engineer Office, Caddoa, Colo.

Analyses of the impervious and pervious materials utilized in the embankment are shown in Figs. 15 and 16. All materials in their natural condition were very dry owing to the arid climate of the region. To get anything approaching the optimum amount of moisture into the material by hosing when spread on the embankment was found to be entirely impracticable.

Consequently, prewetting in the borrow pit was resorted to. In the borrow areas the entire surface was first plowed and then tiny dikes, a foot or so high, were constructed by the use of plow and bulldozer along the contours and water

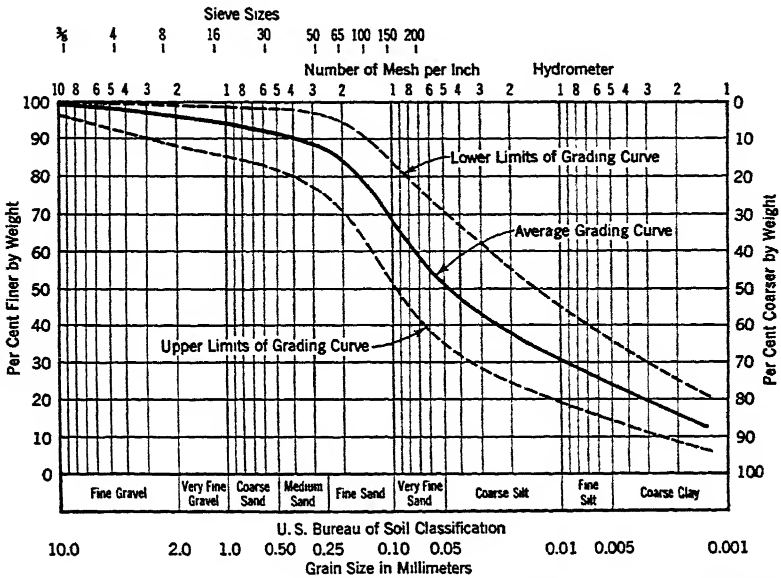

FIG. 15. Impervious Core Analysis, South Dike Conchas Dam. (*From "Design and Construction of Conchas Dam," U. S. Engineer Office, Caddoa, Colo.*)

was kept ponded for about a month over the area. This was generally sufficient to give an average moisture content a couple of points or so below the optimum for a cut of about 25 ft. The necessary additional moisture content was added on the bank as the layers were spread. In general 9-in. loose layers were rolled by the sheep's-foot rollers in about six to eight passes to a thickness of approximately 6 in. Layers were kept high in the center, and when the rare rains appeared they were rolled with plane rollers to shed the water. The dry weight of the impervious fill averaged about 119 lb per cu ft with an average moisture content of 13.2 per cent.

The pervious embankment (Figs. 14 and 16) was spread in layers about 7 in. thick and sprinkled. After three passes of the roller the layers were about 6 in. thick. The average unit dry weight of the pervious embankment was about 132 lb per cu ft, and the average moisture content was about 6 per cent.

As might be expected from the mechanical analyses of Figs. 15 and 16 and

from the dry weight per cubic foot indicated above for the compacted embankment, the shear strength of both the impervious and pervious materials was high. For samples of impervious embankment taken from below the line of saturation, quick shear tests showed a minimum cohesion of 0.15 tons per sq ft with a value for the angle of internal friction (ϕ) of 29°. For the pervious embankment there

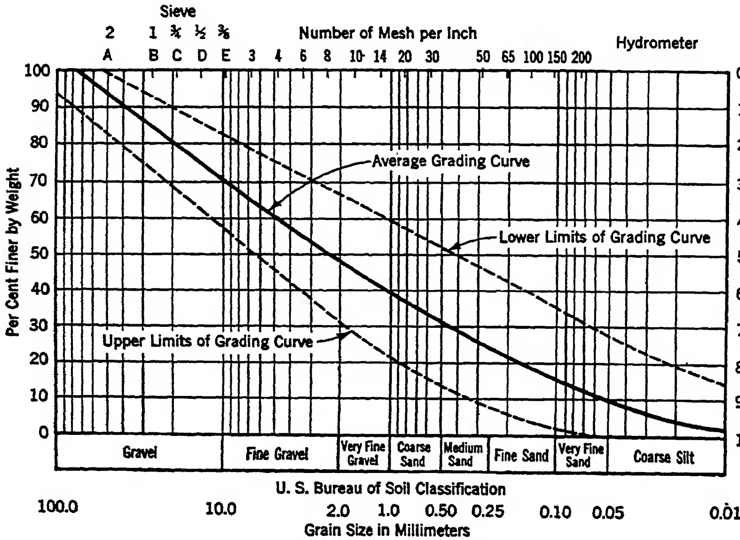

FIG. 16. Pervious section analysis, South Dike, Conchas Dam, New Mexico. *(From "Design and Construction of Conchas Dam," U. S. Army Engineer Office, Caddoa, Colo.)*

was no cohesion and the minimum value of the angle of internal friction (ϕ) was 36°.

29. Clendening Dam. In 1937 the Clendening Dam, located on Brisly Fork, a small tributary of the Muskingum River in Ohio, was constructed as one of the 13 earth dams in connection with the Muskingum Flood Control Project. All these dams were of moderate height (50 to 115 ft), but the sites presented a wide variation in foundation conditions and in materials available for embankments.

The Clendening earth dam was approximately 1000 ft long and 60 ft high. Just as this dam was being completed in 1937, there was a construction accident or partial failure of the dam. At Station 5 + 25, where the greatest movement took place, the top of the dam sagged down approximately 6 ft and the upstream face above the flat toe bulged out a maximum distance of about 5 ft.

Although many reference points were available, not the slightest sign of any movement at either the upstream or downstream toe could be found. There was no movement of the downstream face. Fig. 17 shows a typical cross-section of the dam both as originally designed and as reconstructed after the partial failure. On January 28, 1937, the Board of Consultants advised the placement of additional rock fill on the upstream face in accord with the final

section shown in Fig. 17. At that time the movement in the upstream face was still taking place. The additional rock fill was placed as directed. On February 5, 1937, all movement had ceased, although the placing of this counterbalancing rock fill was not yet complete.

The valley floor which formed the foundation for the dam was approximately 100 ft above ledge rock. The top 40 ft of this foundation was a relatively fat clay with some silt. Below this most of the foundation down to ledge rock consisted of sands and gravels. The 40-ft top layer of clay was a relatively recent alluvial deposit.

The character of this foundation material was about the same as that of the borrow pit for impervious material located just a little way downstream from the

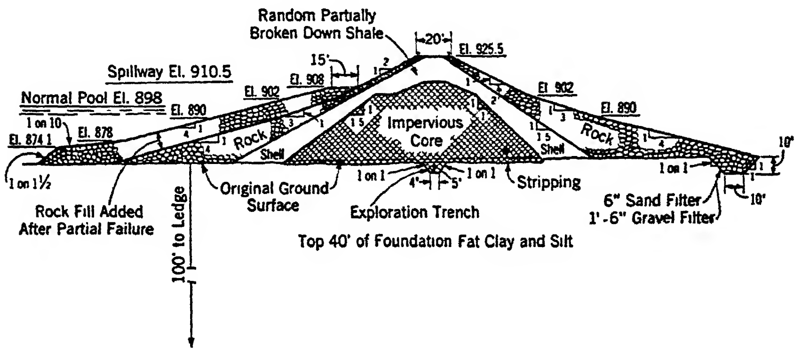

FIG. 17. Clendening Dam, Ohio. (*Courtesy U. S. Engineer Office, Zanesville, Ohio.*)

site. This clay stratum in the foundation before compaction had a moisture content averaging 27.4 per cent, with a maximum of 34 per cent and a minimum of 18.9 per cent.[5]

For the rolled core of the dam the moisture content was found to average 27.3 per cent. The wet weight per cubic foot of this core in the dam was found to be about 122 lb per cu ft, which means a dry density of about 96 lb per cu ft. Both the foundation and the borrow pit for impervious material were subject to frequent overflow. The average clay content of the impervious material was approximately 50 per cent; i.e., 50 per cent of the particles were smaller than 0.005 mm.

In Fig. 17 it will be noted that immediately on the outside of the impervious section is a "random" fill. As intended, this was to consist of shale which was to be spread in thin layers and then broken down by the sheep's-foot roller and compacted to a dense and stable impervious material. Actually over a large part of the dam the breaking down process was imperfect, with the result that much of this random fill consisted of fair-sized particles (¼ in. up to several inches) of shale which when saturated became a slippery, greasy mass which certainly added nothing to the strength of the structure. Above the flow line this mate-

[5] R. R. PHILIPPE, *Soil Studies, Muskingum Conservancy District*, p. 19.

rial was also substituted in the core for the material from the borrow pit described above.

The materials available in the vicinity for building a dam consisted of (1) clays such as those in the borrow pit for impervious material described above, (2) soft shales which rapidly disintegrated on exposure to the weather, (3) hard shales containing sand and lime which resist weathering, and (4) a hard sandstone suitable for riprap or building purposes.

There was no local sand or gravel, so for the concrete and filters of the earth dam this had to be hauled in from many miles away.

In connection with the investigation of the failure, several deep test pits were sunk through the fill and a number of boreholes were put down. As a result of these investigations and the laboratory tests made in connection therewith, it was found:

(1) Neither the upstream nor downstream toe of the dam had moved, (2) the downstream face of the dam had not moved, (3) the upstream face above the flat toe fill had bulged out about 5 ft, (4) the average moisture content of the core material was 27.5 per cent (by dry weight), the wet weight per cubic foot was 122 lb and the dry weight 96 lb, (5) moisture content of the core was the same as when placed, (6) there was no movement in the foundation except that due to settlement and resulting consolidation, which was normal and occurred approximately as anticipated, (7) the moisture content of the random material showed a substantial increase, (8) in the immediate vicinity of the contact between core and foundation there appeared to be a substantial rise in water content, (9) in various parts of the core after the partial failure, slickensides or tiny slip planes were found, (10) tests on undisturbed samples of core after the partial failure showed an angle of internal friction of about 27° with a cohesion value of 0.5 tons per sq ft.

In considering the cause of the partial failure it is significant that the upstream slope of the core was 1 on 1½, whereas the downstream slope was 1 on 1. (See Fig. 17.) In other words the downstream slope of the core was the steeper and consequently there was a materially greater amount of shell material, principally rock, to resist the thrust of the core. Thus this dam was not as safe in an upstream direction against the thrust of the core as it was in a downstream direction. The downstream face did not move at all but the upstream did.

Other Significant Conditions. The core was a relatively fat clay, over 50 per cent of it being of clay sizes. The moisture content, which averaged 27.5 per cent in the material, was high and resulted in a unit dry weight of 96 lb per cu ft and a void ratio of 0.71. There was movement upstream (above toe), with a core slope of 1 on 1½, and no movement downstream, where the slope was 1 on 1.

The impervious core material was placed approximately at Atterberg's plastic limit. (See Arts. 2 and 6, Chapter 16.) It was realized at the time that the core material was wetter than desirable, but it was the only material available.

For the given moisture content the amount of rolling was excessive and may have resulted in the over-compaction of the core material with consequent

imprisoning of expansive forces in the rolled material. It might also be stated that for the compaction utilized the moisture content was excessive.

In view of the above conditions and in utilizing hindsight it is now evident that satisfactory stability of the section could have been attained either by steepening the slope of the upstream face of the core from 1 on 1½ to 1 on 1 or even steeper or by retaining the slope of the core and adding additional rock fill on the upstream face.

The latter method was that chosen by the Board of Consultants for stabilizing the section after the partial failure. The source of most of the data in relation to the Clendening Dam partial failure is a very interesting unpublished report by R. R. Philippe, supplemented by personal observation.

30. Contents of Earth Dams. In connection with preliminary investigations it is frequently necessary to make a number of preliminary estimates of cost of earth dams. Accordingly, in Table 1, there are given a number of formulas for finding the volume of earth dams for various heights and slopes. Figs. 18 and 19 give the same data by the use of curves.

TABLE 1

Formulas for Volume of Earth Dams, in Cubic Yards per Linear Foot *

[Top width equals 0.25 of the height of the dam]

Slope of One Face	*Slope of Other Face*	*Volume*
1 on 2	1 on 2	$0.0833h^2$
1 on 2.5	1 on 2	$0.0926h^2$
1 on 2.5	1 on 2.5	$0.1021h^2$
1 on 3	1 on 2	$0.1020h^2$
1 on 3	1 on 3	$0.1203h^2$
1 on 3.5	1 on 2	$0.1110h^2$
1 on 3.5	1 on 3	$0.1296h^2$
1 on 4	1 on 2	$0.1203h^2$
1 on 4	1 on 2.5	$0.1295h^2$
1 on 4	1 on 3	$0.1387h^2$
1 on 4	1 on 4	$0.1574h^2$
1 on 5	1 on 5	$0.1944h^2$
1 on 6	1 on 6	$0.2313h^2$

* h equals height of dam.

31. List of Earth Dams. In Table 2 is given a list of representative earth dams. This list is not comprehensive, but it does include a considerable proportion of the more notable structures. It will be noted that it includes a number of the dams which suffered a partial failure during construction but which were reconstructed and have fulfilled their function satisfactorily ever since. References to published articles on the dams listed are given, and the list is sufficiently representative so that if one were to study the published data for all of them, he would obtain a fair cross-section of the present status of the science and art of the design and construction of earth dams. The list was prepared by S. A. Sutherland, Designing Engineer, Harza Engineering Co.

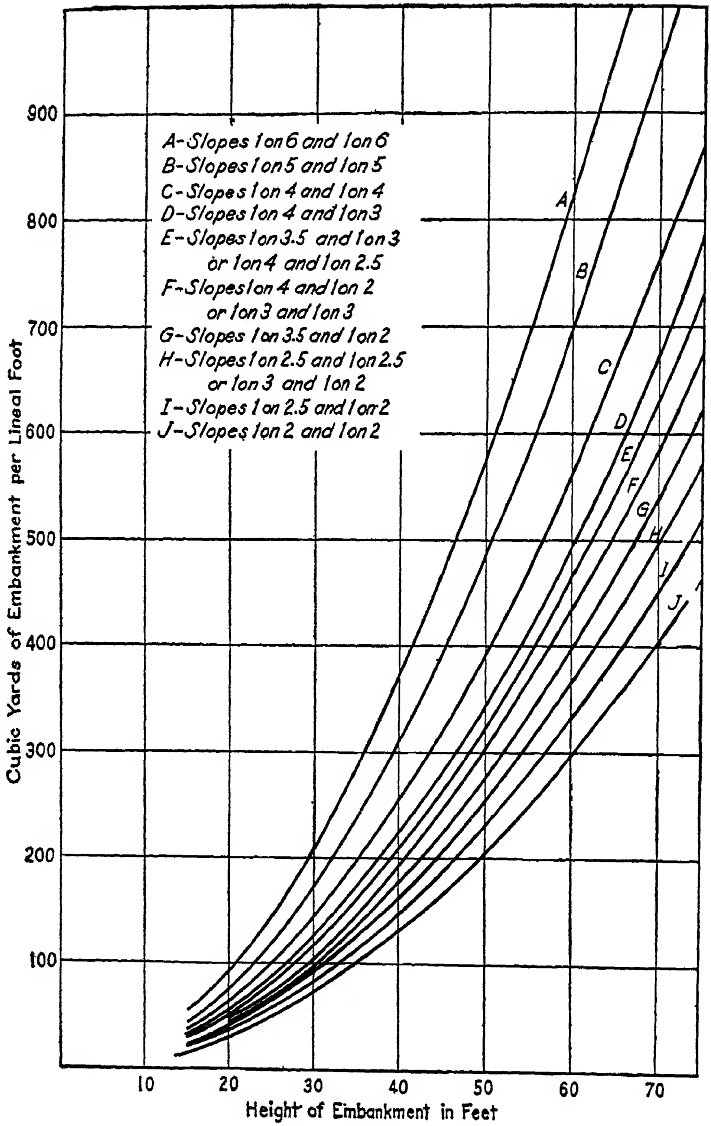

FIG. 18. Contents of earth dams for various heights and slopes. Top width = 0.25 height; heights, 15 to 75 ft.

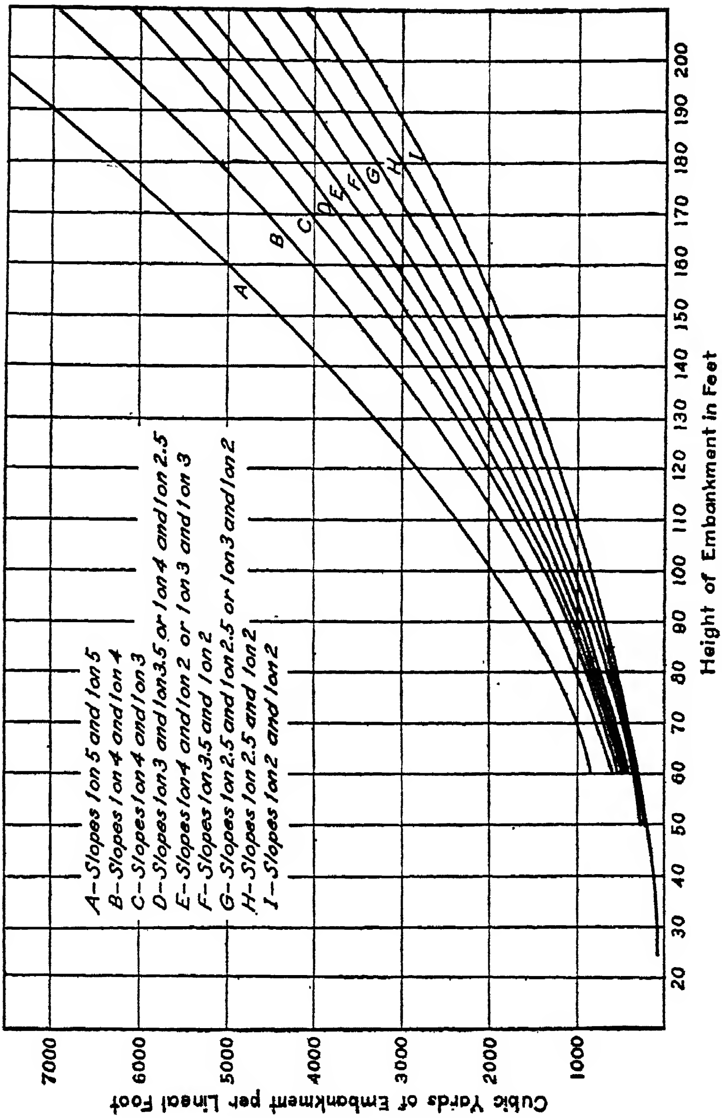

Fig. 19. Contents of earth dams for various heights and slopes. Top width = 0.25 height; heights, 60 to 210 ft.

TABLE 2

SOME REPRESENTATIVE EARTH DAMS

[Prepared by Robert A. Sutherland]

(1) Name	(2) Location	(3) Height (ft)	(4) Built	(5) Upstream Slope	(6) Downstream Slope	(7) Wave Protection	(8) Fetch (Miles)	(9) Free-board (ft)	(10) Method of Con-struction	(11) References	(12) Remarks
Alamogordo	N. Mex.	135	1938	5, 3	$2\frac{1}{2}$	3′ rr	7	10	rf	Y 6/38	
Alcova	Wyo.	180	1938	3	8, 2	rr	. .	10	rf	Y 8/38	
Alexander *	Hawaii	140	1932 *	3	$2, 1\frac{1}{2}$	rr	. .		hf	P 10/20/32	
Arkabutla	Miss.	67	1941	$3\frac{1}{2}, 3, 2\frac{1}{2}$	$3, 2\frac{1}{2}$	2.5′rr	. .	8	rf	17	
Apishapa	Colo.	120	. . . .	3	2					g 3/27	Conc. core wall.
Ashokan Dike	N. Y.	110	1914	$2\frac{3}{4}, 2$	3, 2	st, rr		20	rf	3, 13	
Azucar, El.	Mexico	138	1939	6, 3	6, 2	3′ rr		10	rf	P 8/25/38	
Belle Fourche *	S. Dak.	115	1909 *	3, 2, 1	$2, 1\frac{3}{4}$	cs		15	rf	3, 16	
Bills Brook	Conn.								rf	K 9/36, P 9/12/35	Conc. core wall.
Birch Hill	Mass.	44	1940	$5, 4, 2\frac{1}{2}$	$5, 4, 2\frac{1}{2}$	3′ rr	6	12	rf	19	
Blue Ridge (Catawba)	Ga.	170	1919						hf	P 12/10/31	
Boca	Nev.	110	1940	6, 3	$8, 2\frac{1}{2}$	rr	3	7	rf	g 9/38	
Bouquet Canyon	Calif.	213	1934	3	3	6″–9″ rc		10	rf	G 8/34, P 6/21/34	
Cajalco	Calif.	210	1938	3	$3, 2\frac{3}{4}$	8″ rc	2	14	rf	G 6/38	
Calaveras *	Calif.	215	1925 *	$3\frac{1}{2}, 3$	$2\frac{1}{2}$	cs			hf	16	
Cle Elum	Wash.	135	1932	20, 4, 3	8, 3	2.5′ rr	4.5	10	rf	2	
Cobble Mountain	Mass.	235	1932	$5\frac{1}{2}, 3\frac{1}{4}, 2\frac{1}{4}$	$5, 3\frac{1}{4}, 2\frac{3}{4}, 2\frac{1}{4}$				hf	i, pa 243, Vol. 99, 1934	Clay core.
Conchas Dikes	N. Mex.	100	1937	3	3	rr	12	5	rf	18	
Conklingville (Sacandaga)	N. Y.	100	1930	$4, 3\frac{1}{2}, 3$	$1\frac{1}{2}, 2, 4\frac{1}{4}, 2\frac{1}{2}$	rr		[a] 24	hf	P 7/30/31	
Cummum	India	102	1900	$3, 1\frac{1}{2}$					rf	3	Conc. core wall.
Deer Creek	Utah	155	1942	5, 3	$2, 6, 2\frac{1}{4}$	3′ rr	5	8	rf	G 1/39	
Denison	Texas	165	1942	$4, 2\frac{1}{2}$	3, 2	st, rr			rf	P 5/25/39, 6/6140 9/19/40	
Dodder	Ireland	115	1900	3	3				rf	5	

Don Martin (Nueva Espana)	Mexico	105	1931	$1\frac{3}{4}$	2	8″–11″ rc		13	rf	i, 1931	
Druids Lake	Md.	119	1871	4	2	st, rr	small	5	rf	16	Clay puddle core.
Echo	Utah	125	1929	5, 3	$1\frac{1}{2}$, 6, 2	4′ rr	4, 5	10	rf	2	
El Capitan	Calif.	217	1934	3, $2\frac{1}{2}$, 2	3, $2\frac{1}{2}$, 2	rr		a 20	hf	P 7/13/33	
Encino	Calif.	127	1924	2	2					g 3/27	rc core wall.
Fort Peck *	Mont.	242	1940 *	4	$8\frac{1}{2}$	rr	25	20	hf	16	Steel cutoff.
Franklin Falls	N. H.	136	1942	3	3	rr			rf	P 6/16/38	
Gatun	Canal Zone	115	1912	7.67, 4	8, 16, 8, 4				hf	16	
Goose Creek	Idaho	145	1913	3	2						
Grassy Lake	Idaho	120	1939	6, 3	4, $2\frac{1}{2}$	3′ rr	1	8	rf	Y 12/39	
Green Mountain	Colo.	270	1942	3	5, $2\frac{1}{2}$	3′ rr		10	rf	P 1/5/39, g 3/41	
Guernsey	Wyo.	105	1928	3	8, 2	3′ rr	3	10	hf	2, P 2/16/28	
Hansen	Calif.	100	1941	3	6, 5, 3	12″ rr		a 27	rf	P 11/7/40	
Harriman (Davis Bridge)	Vt.	200	1923	4, $3\frac{1}{2}$, 3	$3\frac{1}{2}$, 3, $2\frac{1}{2}$		5	30	shf	21	
Hume	Australia	142	1935	3, $2\frac{1}{2}$	$2\frac{1}{2}$, 2				rf	3	Conc. core wall.
John Martin (Caddoa)	Colo.	130	1941	3, $2\frac{1}{2}$	3, $2\frac{1}{2}$, 2	3′ rr		10	rf	P 11/7/40	
Kingsley (Keystone)	Nebr.	160	1941	3, 3, 2	$2\frac{1}{4}$, 3, $3\frac{1}{4}$, 4	Conc. rr	21	12	hf	8/10/40, P 7/14/38 7/21/38	Steel cutoff.
Knightville	Mass.	143	1940	3, $2\frac{1}{2}$	3, $2\frac{1}{2}$	4′ rr	3.5	20	hf	19	Hydr. fill core.
Lahontan	Nev.	129	1915	3	2				rf	2, 8	
Little Bear Creek	Calif.	200	1922	$2\frac{1}{2}$	2					13	Conc. core wall.
Long Valley	Calif.	182	1941	3	3	rr		15		g 5/39	
McKay	Oreg.	165	1928	$1\frac{3}{4}$	2	cs	4	8	rf	2	
Merriman (Lackawack)	N. Y.	200	1941	4, $3\frac{1}{2}$, $2\frac{1}{2}$	4, 3	rr	7	20	rf	P 11/21/40	Caisson cutoff.
Mohawk	Ohio	115	1936	3, 2	$3\frac{1}{2}$, $2\frac{1}{2}$, 2	rr		a 20	rf	I 12/36	
Mohicanville	Ohio	53	1936	$3\frac{1}{2}$, 3	$3\frac{1}{2}$, 3	rr		15	rf	20	
Morris	Conn.	100	1910	3, $2\frac{1}{2}$	2			9	rf	5	Conc. cutoff.
Mudduk	India	108	1500	Base	1100 ft					16	
Mud Mountain (Stevens)	Wash.	425	1942	2, $1\frac{1}{2}$, $1\frac{3}{4}$	$2\frac{1}{4}$, $1\frac{3}{4}$	rr	5.5	a 35		P 3 3/14/41, g 8/39	Treated earth core.
Necaxa *	Mexico	180	1909 *	3	2			16	hf	8, 16	
New Croton	N. Y.	122	1906	2	2				rf	13	Rubble masonry core wall.

[Footnotes and abbreviations are given on page 781.]

TABLE 2—*Continued*

SOME REPRESENTATIVE EARTH DAMS

[Prepared by Robert A. Sutherland]

(1) Name	(2) Location	(3) Height (ft)	(4) Built	(5) Upstream Slope	(6) Downstream Slope	(7) Wave Protection	(8) Fetch (Miles)	(9) Free-board (ft)	(10) Method of Construction	(11) References	(12) Remarks
Niederwartha	Germany	138	1929	$3\frac{1}{2}$	4, 3, 2	rr		7	rf	P 7/30/31	
Piedmont	Ohio	56	1936	$3\frac{1}{2}$, 3, 2	$3\frac{1}{2}$, 3, 2	1 5′ rr		13	rf	20	
Prado	Calif.	105	1941	3, $2\frac{1}{2}$	6, 5, $2\frac{1}{2}$	12″ rr		10	rf	g 10/38, 12/39, 2/41	Steel cutoff.
Priest	Calif.	147	1923	$2\frac{1}{2}$	2			5	Hydr. dump	G 5/34	Conc. core wall.
Puddingstone No. 1	Calif.	166	1927	$2\frac{1}{2}$	$2\frac{1}{4}$					g 3/27	
Quabbin Dike	Mass.	135	1938	$2\frac{1}{2}$, 3, $1\frac{1}{2}$, 2	$2\frac{1}{2}$, 2	rr		20	hf	P 8/24/33, 1/24/35, 6/18/36	Caisson cutoff.
Ralston Creek	Colo.	200	1938	4, 3	2	1.5′–2.5′ rr		[a] 34	rf	P 7/29/37	
San Leandro (lower)	Calif.	135	1876–98	3	$6\frac{2}{3}$			5	rf and hf	16	
Santiago Creek	Calif.	130	1932	$2\frac{1}{2}$, $1\frac{1}{2}$	2, $1\frac{1}{2}$	6″ rc				P 5/12/32	
Sardis	Miss.	117	1940	15, $3\frac{1}{4}$, $2\frac{3}{4}$	7, $4\frac{1}{2}$, $3\frac{1}{4}$	rr		[a] 30	hf	G 6/39, P 3/16/39	
Senecaville	Ohio	47	1936	5, 3, $2\frac{1}{2}$	5, 3, $2\frac{1}{2}$	rr	7	13	rf	20	
Silvan	Australia	145	1931	3	2	12″ rr			rf	P 9/3/31	Conc. core wall.
Standley Lake *	Colo.	147	1912 *	3, 2	2	cs on 3′ grav.		5	rf	13, 16	Puddle core.
Surry Mountain	N. H.	74	1939	5, 3, $2\frac{1}{2}$	5, 3, $2\frac{1}{2}$	3′ rr	$2\frac{1}{4}$	15	rf	19	
Tabeaud	Calif.	123	1902	3, $2\frac{1}{2}$	$2\frac{1}{2}$	rr		8	rf	16	
Taylor Park	Colo.	168	1940	3	4, 2	3′ rr		14	rf	2	
Temescal	Calif.	115	1868–86	3	5			5	hf and rf	5, 16	
Terrace	Colo.	180	1909	3	2				hf	16	15′ Conc. core, part hf.
Tieton	Wash.	223	1924	3	2	4′ rr		9		2	Conc. core wall.
Titicus	N. Y.	102	1895	2, 4	$2\frac{1}{2}$					13	Rubble core wall.
Vallecito	Colo.	125	1940			3′ rr			rf	P 6/30/38	
Wachusett N. Dike	Mass.	100	1906	2	11, 5, $2\frac{1}{2}$	3′ rr on 7′ grav.		17	rf	5, 16	

Winsor (Quabbin Res.)	Mass.	170	1939	$2\frac{1}{2}$, 3, $1\frac{1}{2}$, 2	$2\frac{3}{4}$, $2\frac{1}{2}$, 2	rr		20	hf	P 5/19/38	Caisson cutoff.
Yarrow	England	100	1905	3	2	2′ rr		7	rf	16	Clay puddle core.

Notes: Slopes given as horizontal distance to unity vertical.

Core walls noted when an important part of structure. Minor cutoff walls not noted.

* indicates some degree of failure during or after construction, but in all cases successful repairs were made.

[a] Freeboard given above spillway crest.

Abbreviations: *Wave Protection:*

rr riprap (stone).
cs concrete slab.
rc reinforced concrete.

Method of Construction:

rf rolled fill.
hf hydraulic fill.
shf semi-hydraulic fill.
d dumped.

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16. Wegmann, "The Design and Construction of Dams," 1927.
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See also *Earth Dam Projects*, Table 14, p. 220.

32. Hydraulic and Semi-Hydraulic Fill Dams. Because of the magnitude of some hydraulic fill dams, great publicity has been given to the several slides and construction accidents which have occurred in connection with the construction of several of them. As a result some engineers exhibit a prejudice against that type of construction. As a matter of fact, since 1930 there have been several more or less serious construction accidents in connection with rolled fill earth dams but only one in connection with a hydraulic fill dam. There is no sound reason for such prejudice. All that is necessary is for the engineer to appreciate that the hydraulic fill dam is an engineering structure and that it should be given the same competent attention in investigation, design, and construction that one would give any other engineering structure.

Hydraulic fill and semi-hydraulic fill dams must fulfill the same criteria of design as earth dams, which are built by depositing material in layers and rolling, but, by reason of the methods of construction used, they offer an entirely different construction problem.

The following definitions [6] for hydraulic fill and semi-hydraulic fill dams are generally accepted.

Hydraulic Fill Dam. An earth dam in the construction of which the materials are transported onto the dam by water and distributed to their final position in the dam by water is a hydraulic fill dam.

Semi-Hydraulic Fill Dam. An earth dam in the construction of which the materials are transported onto the dam and dumped within the section of the dam by some other means than water, but some of this material is moved to its final position in the dam by the action of water is a semi-hydraulic fill dam.

The semi-hydraulic fill method of construction has sometimes been the cheapest and most convenient for use at a given site. There are, however, certain dangers generally inherent in this method of construction which should be appreciated and guarded against if the method is adopted. Dams built by the hydraulic fill method have been comparatively free from slides during or immediately after construction,[7] whereas several dams constructed by the semi-hydraulic fill method have had slides during construction.

A fundamental difference in stability during construction thus seems to be indicated. The hydraulic fill method deposits the material from flumes or pipes near the faces of the dam; the larger particles stay there and the finer ones move toward the center, the finest of all going into the central pool and being deposited there. Thus the toe and faces of a dam produced by this method are more pervious, allowing water to drain out from the interior of the dam. Even if most of the drainage from the cores is by vertical crater action, as some engineers maintain, such action takes place not only in the portion of the core underlying the central pool but also in those portions of the core covered by the pervious outer sections of the dam. Hence, whether the main drainage of the core is upward, downward, or sidewise, the importance of pervious outer sections is just as great.

[6] *Trans. Am. Soc. Civil Engrs.*, Vol. 85, 1922, p. 1215.
[7] The Alexander Dam (*Earth Dam Projects*, p. 7) is a notable exception to this statement.

In dams built by the semi-hydraulic fill method, the toes and faces usually have consisted of car dump fills. Material is washed away from these fills by jets of water from giants. The finer material goes into a central pool and is deposited, forming the core; the coarser particles are dropped near the car dump fill. In consequence of this action, the car dump fill at the face is often more dense and impervious than the material immediately adjoining it on the inside of the dam, for this latter material has had the fines washed out of it by the action of the monitors.

In some cases tests have shown that the material in the car-dumped fills is actually more dense and impervious than that immediately adjoining. Through the presence of the central pool and the sluicing operations, this comparatively

Fig. 20. Typical double-jointed ball-bearing giant for sluicing.

pervious area is kept full of water, which exerts considerable hydrostatic pressure on the relatively impervious car fill material at the face. Thus the car fills may form an element of weakness and by imprisoning the water may sometimes be the cause of slides even though the central core of fine material is so stable that it exerts no hydrostatic pressure.

Engineering and economic factors have narrowed the possible use of semi-hydraulic fill dams so that the situations in which they might be used have become rare indeed. Consequently, very little space is devoted to them herein.

The design and stability of hydraulic fill dams has been considered in Arts. 21 to 25, Chapter 18.

In Fig. 22 is shown the typical cross-section of the five Miami dams in Ohio. Fig. 23 is a cross-section of Sardis Dam, Mississippi, and Fig. 24 is a typical cross-section of Cobble Mountain Dam in Massachusetts. These hydraulic fill dams are more or less typical of many others.

33. Typical Hydraulic Fill Construction. In typical hydraulic fill dams construction water under heavy pressure is delivered to a "giant" or "monitor," which is a large-sized nozzle mounted on a standard with a ball and socket joint

so that the direction of the stream may be readily changed without moving the standard, which is generally weighted down with stones or iron while in operation. The giant is directed against the bank which it is desired to excavate in such a manner that the water will undercut the material and then break it up. The velocity of the water as it leaves the giant may be from 100 to 200 ft per sec. As the force of the water cuts down the material, the water and loosened material are guided to flumes or pipes through which they flow to the dam and are deposited.

The minimum grade of these flumes or pipes is from 3 to 6 per cent according to the nature of the material. The water for the giant is obtained from high pressure pumps located at the river side, or else from a pipe line with its source at

FIG. 21. Hydraulic giants operating in borrow pit, Almanor Dam, North Fork, Feather River, California. (*Courtesy Pacific Gas & Electric Co., San Francisco.*)

a point having a much higher elevation. In some cases the material is excavated by suction dredges and pumped to the site through pipe lines.

Flumes or beach pipes are generally maintained at both the upstream and downstream faces, discharging water and materials toward the center of the dam. A central pool of water is maintained whose width varies constantly between maximum and minimum widths prescribed directly or indirectly by the requirements of the engineer, as will be shown. As the water discharges from the sluices or trestles, the coarser material is deposited near the faces and the finer material flows into the central pool with the water and is slowly precipitated. In general, there is a gradation from the coarsest material near the faces of the dam to fine sand at the shoulder of the core pool with the extremely fine material being deposited in the core pool. In many cases, however, there is some of the coarsest material deposited near the faces, but the body of the shell is a mixture

without the nice gradation of sizes which should theoretically be found in the shell from coarse to fine as the core pool is approached.

In Fig. 26 is shown the typical operation of a core pool in the building of a dam by hydraulic methods. In order to make the illustration concrete, it will be

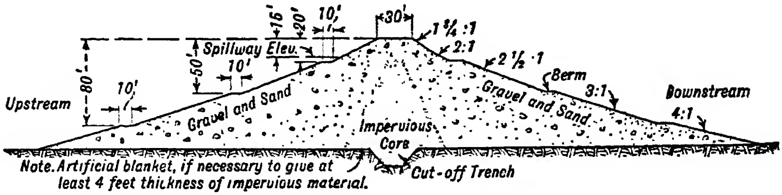

FIG. 22. Typical section of hydraulic fill dams of Miami Conservancy District. (*From "Core Studies in Hydraulic Fill Dams of the Miami Conservancy District,"* by *Charles H. Paul, Trans. Am. Soc. Civil Engrs. Vol. 85, p. 1181.*)

assumed that the lift used is 5 ft and that the slope which the beaches will take with the given materials and method of operation is 5 per cent (1 on 20). A 5-ft lift is obtained if we go across the dam discharging material from the pipes on both sides and removing or putting on the beach pipe section by section (or open-

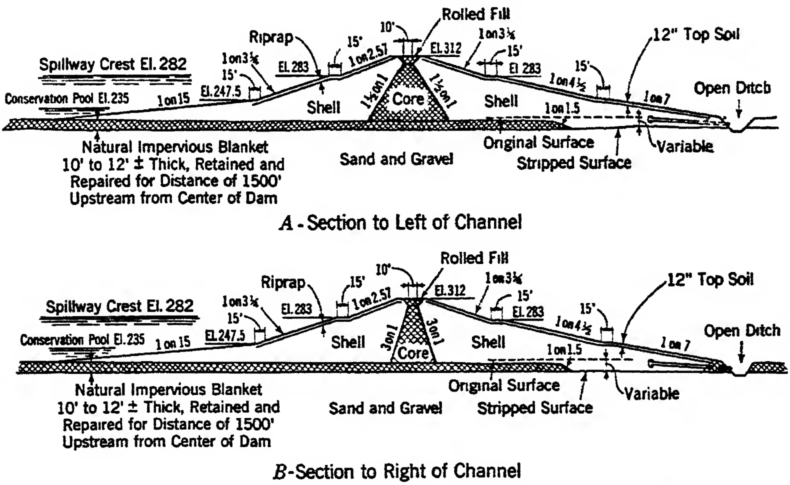

FIG. 23. Sardis hydraulic fill dam, Mississippi. (*Courtesy U. S. Engineer Office, Vicksburg, Miss.*)

ing and closing the windows or traps in pipes) when 5 ft of material has been deposited on the beach at any point.

Before the beginning of the construction of this particular lift, the beach surface and the core surface are as shown in Fig. 26 by line *ABCD*. It will be noted that the surface of the core is 5 ft below the shoulder point at *B*.

The water surface of the pool is then raised until it is 5 ft above shoulder point *B* in elevation, and at that time the pool necessarily extends out to a point *N*

135 ft from the center line as shown because the beach slope is 5 per cent. As a result the depth of the core pool at this instant and at this station is 10 ft. Then the discharge of material is started from the beach pipes and the beach begins to build up and the edge of the pool is crowded closer to the center line.

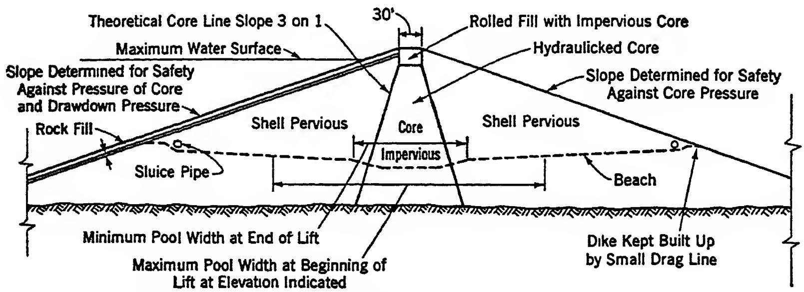

FIG. 25. Method of construction of typical hydraulic fill dam.

This procedure continues at the given station with the fines going into the pool and being deposited until the pool is crowded in by the deposit of beach material to point B_1, which is the position of the new shoulder stake. At this point the sluicing is stopped at this particular station and the procedure is continued at another station until the lift has been carried all the way across the dam.

On completion of the lift, the beach surface and core surface are indicated in Fig. 26, by the line $A_1B_1C_1D_1$. At this instant the edge of the core pool is at the shoulder B_1, approximately 70 ft from the center line of the dam and the depth

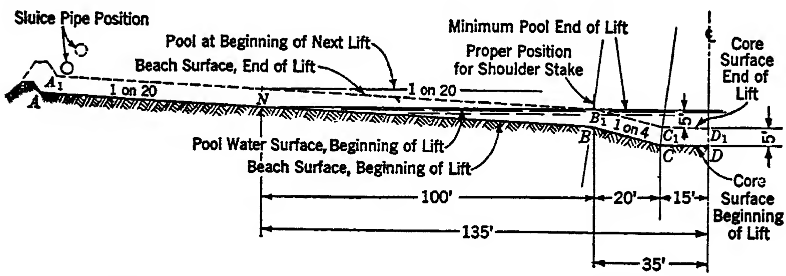

FIG. 26. Typical operation of core pool for 5 ft lifts on hydraulic fill dam. (See Art. 33.)

of the core pool is again 5 ft at this particular station. At other stations the depth of the pool will range from 5 to 10 ft according to the stage in the operation.

While in all hydraulic fill dams the material is carried on to the dam by water and deposited, there are several different ways that the material may be excavated and transported before it reaches the dam. At the borrow pit it may be excavated by hydraulic giants, by dredges, by steam shovels, or by drag lines. In the latter two cases the material is transported dry to a hog box where it is mixed with water and then run or forced through pipes to the beach pipes on the dam.

Thus at the Winsor (Quabbin Dam) in Massachusetts, material in a borrow pit was excavated by Diesel operated shovels and deposited directly through hoppers onto a movable belt conveyor system which a succeeding belt elevated to the top of a hill overlooking the dam. Here it was deposited into a hog box and thoroughly mixed with water by the action of the jets, after which the mixture 15 to 22 per cent of which was solids, flowed by gravity to the beach pipes on either side of the core pool. For stability of hydraulic fill dams see Art. 21, Chapter 18.

34. Some Details of Hydraulic Fill Construction.

Required Velocity in Sluice Lines. One of the most important factors in any sluicing operations is the velocity at which the mixture of water and material is transported. If the velocity in the sluice line is too low the material will drop out and deposit in the pipe, resulting in frequent stoppages. Frequently this will require the disjointing and cleaning out of the line before work can be resumed. Fortunately there is some range between a velocity which is definitely satisfactory and one which will cause serious stoppage.

This is due to the fact that as the average velocity over the cross-section of the pipe decreases and causes material to drop out, the effective cross-section is thus reduced, and this in turn tends to increase the velocity over the remaining cross-section and thus prevents additional material from being deposited. Once a definite deposit is built up to make the pipe about half full, the required increase in velocity to make the lodged material start moving again is frequently so great that the only practicable way to restore the capacity of the line is to disjoint it, clean it out, and start afresh.

The velocity which will prove satisfactory varies with the size of the pipe, the character of the material being transported as clay, sand, gravel, angular gravel, and rock and also with the percentage of solids. Other things being equal the minimum satisfactory velocity will be less for a small pipe than for a large one.

In a rough way the following mean velocities may be utilized as safe minimum velocities in sluice lines carrying a 15 per cent mixture [8] of sand and gravel.

16-in. pipe	15 ft per sec
18-in. pipe	16 ft per sec
20-in. pipe	18 ft per sec
24-in. pipe	22 ft per sec
30-in. pipe	24 ft per sec

Friction Loss. In making hydraulic computations for a sluicing system consideration should be given to the fact that we are dealing with a liquid heavier than water. The unit weight is dependent on the percentage of solids and may be readily determined for any given case. In general, unit weights found will be between 70 and 80 lb per cu ft. If the line is operating under gravity conditions, this helps to compensate for the additional friction over what would occur

[8] A 15 per cent mixture as herein used means that 15 per cent by volume of the mixture would be earth in borrow pit measure, i.e., in 100 cu ft of mixture there would be 85 cu ft of water and 15 cu ft of material as measured loose in the borrow pit.

with plain water. On the other hand, if it is a pump line, the heavier unit weight of the liquid in effect results in having a higher effective head to pump against.

The mixture of material and the percentage of solids has a great effect on the friction head. Thus a mixture of glacial sand and gravel with 10 per cent fat clay will show a rather small friction factor, whereas the same mixture without the clay might show a relatively high friction factor. Everything considered, for rough computations, it is suggested that the Hazen Williams formula with $C = 75$ be used for sand gravel mixture without lubricating clay, and $C = 100$ with lubricating clay.

Thus $V = 75 \times 1.32R^{0.63}S^{0.54}$

in which V = mean velocity of mixture in pipe,
R = hydraulic radius = cross-section area divided by wetted perimeter,
S = slope (as 0.01 for a slope of 1 ft per hundred).

It is best to have the sluice line on as even a grade as practicable avoiding humps and hollows in the line.

Sluicing Box. The sluicing box (sometimes called hog box) can become the bottle-neck of the sluicing operation. The dry materials dumped into the box must be thoroughly saturated and suspended in the water before it leaves the sluicing box and enters the sluice line. Fig. 27 indicates the arrangement of a

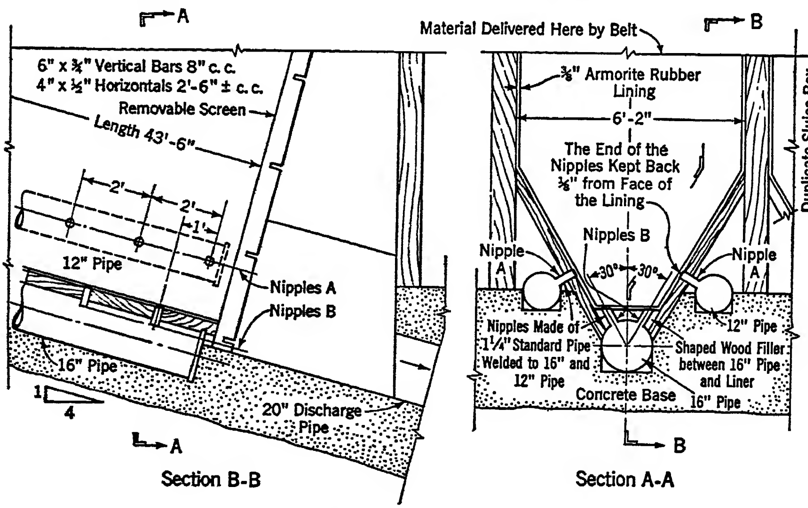

FIG. 27. Sluice box at Winsor Dam, Massachusetts.

successful sluice box at Winsor Dam, Massachusetts. This was a double box, one side normally feeding the downstream sluice line and beach pipes and the other the upstream sluice line and beach pipes.

It will be noted that the bottom of the sluice boxes is almost triangular and is equipped with a number of jets for saturating and agitating the material dumped

into the box. The box was fully lined with rubber, which minimized wear and noise. At the back end of the sluice box there were booster nozzles which could be utilized to push the material along into the entrance of the sluice pipes. The normal pressure of the jets was 10 to 15 lb per sq in., but a booster pump was available to raise the pressure of the water jets by 50 lb to the sq in. if required.

At the back and above the twin sluice boxes was a control room overlooking the sluice box, the head of the belt conveyor, and the beaches. By means of controls on the hydraulically operated valves and remote control of motors, one operator could handle the elevating of the material to the sluice box, its mixing with water, and its delivery to the beaches. The plant was designed for 750 cu yd per hr, but maximum capacity was over 1000 cu yd. The capacity of water line and pumps to the sluice boxes was 25,000 gpm. As the mixture was 16 to 22 per cent solids, the water capacity was found to be ample, with some reserve capacity.

Beach Pipe. Pipe instead of open flumes are now generally used for discharging material near the outer limits of the beaches (see Fig. 25). The method of operation is either end discharge or trap (window) discharge. End discharge is applicable to beach pipe lines up to and including 20-in. pipe. Trap discharge is applicable to 20-in. beach pipe lines and larger.

With end discharge the beach pipe line is laid all the way to the far end of the beach. With the line discharging then at the end the lift is filled out at this point. Then a section of pipe is removed and filling is repeated, and so on until all the pipe has been disjointed. Next the beach crew adds on a section of pipe, fills the lift, and adds another section of pipe, and so on across the length of the beach.

Throughout the entire procedure of taking off and putting on pipe the discharge of the mixture of material and water is never willingly stopped. A small caterpillar tractor crane is used at the outer edge of each beach to handle the pipe and to keep the outer levies built up. For beach pipe up to about 20 in. in diameter this end discharge method is generally the most convenient.

When trap (or window) pipe is used the beach line is not often disjointed but is kept in position and jacked up and moved in a bit for succeeding lifts. In some cases the pipes are merely supported on blocking and are jacked after each lift; in other cases trestles support the pipe high enough so that several lifts may be run before another trestle is installed and the pipe disjointed and set up again.

The size of the trap is important. There is a usual tendency to make it too small. It should be one diameter wide and 1½ diameters long in order to provide the best control.

The traps should be either on the bottom or the side of the pipe, whichever works best under the given conditions. Doors for opening and closing traps should be hinged in such a manner that they move away from the opening and do not interfere with the discharge. Traps should be located not farther apart than one in every other 16-ft length of pipe or they may be located in short sections of pipe inserted between each two 16-ft lengths.

The trap method is certainly more convenient for 24- to 30-in. beach lines and may be desired for 20-in. lines. A number of traps are opened at a time and when the lift opposite a trap is completed that trap is closed and another is opened. Operating of traps should be in sequence instead of jumping around

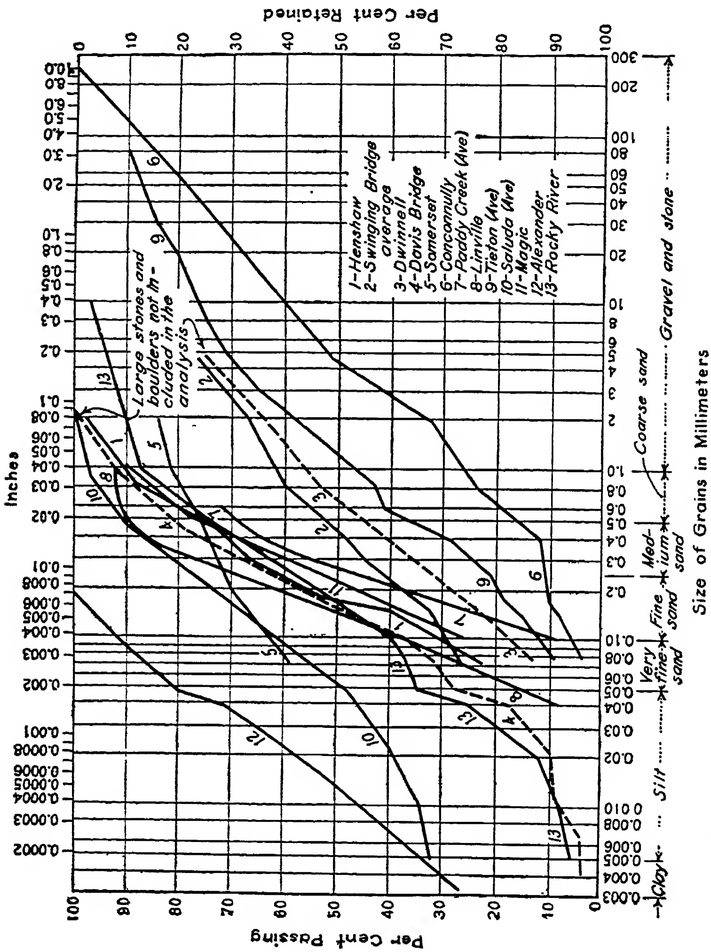

Fig. 28. Mechanical analyses of borrow pit materials of hydraulic and semi-hydraulic fill dams.

from one part of the dam to another as this will help in maintaining an even beach line, which is essential in minimizing the development of tongues of sand into the core and tongues of core into the shell. Shear boards are also utilized on the beach to help to maintain the beach line.

Wear of Sluice Pipe. The wear on the sluice pipe varies greatly with the character of the solids carried. For instance, a 3/8-in. thick 20-in. pipe of ordinary grade may not be good for more than 700,000 cu yd of sand gravel and

stones up to 6 in. in size if many of the particles are angular. On the other hand, the same pipe carrying a mixture of sand, gravel, rounded stone, and some clay may carry several times the above yardage before it has to be discarded.

The use of steel containing 0.75 per cent of manganese with about 0.6 per cent of carbon has been found to materially increase the life of the pipe.

At Winsor (Quabbin Dam) 20-in. pipe lined with rubber was used because of the abrasive nature of the material. Some of this rubber-lined pipe carried in excess of 2 million cu yd. but only a small percentage of the rubber lining was worn out.

35. Materials Suitable for Hydraulic Fill Dams. Not all materials are suitable for use in the construction of a hydraulic fill dam. Fine materials which are very uniform in size should not be utilized for the construction of hydraulic fill dams. Such materials are apt to be deposited in the shell at a density less than that at critical void ratio. (See Art. 13, Chapter 16.) In this condition a uniform material would be subject to a flow slide. Also soils composed almost exclusively of very fine particles, as clays and silts, or silts and fine sands, cannot be used in making hydraulic fill dams as the particles will not settle and consolidate promptly enough.[9]

Accordingly, it is desirable that the materials in the shells of hydraulic fill dams should be nonuniform in character. The uniformity coefficient (see Art. 2, Chapter 16) is one index to the desirability of the material in the shells of a hydraulic fill dam. The higher the coefficient the more nonuniform the material. If the coefficient were unity all particles would be the same size. It is not by any means a complete index of desirability, but it is nevertheless a useful guide. The following are uniformity coefficients of typical shell material of the following dams:

Dam	*Uniformity Coefficient of Shell*
Sardis, Mississippi	4.5
Winsor, Quabbin, Mass.	16.7
Kingsley, Nebraska	5.4
Fort Peck, Montana	3.0
Knightville, Massachusetts	8.2
Quabbin Dike, Massachusetts	15.0
Wichita Falls, Texas	2.8
Garza, Texas	3.7
Germantown Dam, Ohio	16.6
Englewood, Ohio	14.0

Glacial deposits frequently provide materials which are ideal for the construction of hydraulic fill dams. The materials ranging all the way from clay and rock flour sizes up to stones and boulders too large to be conveniently transported by hydraulic methods. Cobble Mountain and Winsor Dams in Massa-

[9] For the story of an unsuccessful attempt to build a hydraulic fill dam almost exclusively of very fine material, see Joel D. Justin, *Earth Dam Projects*, p. 7 (Alexander Dam), John Wiley & Sons, Inc., New York, 1932.

chusetts, the Miami Dams in Ohio, and Kingsley Dam in Nebraska are all built of glacial sands, gravel, stone, silts, and clays.

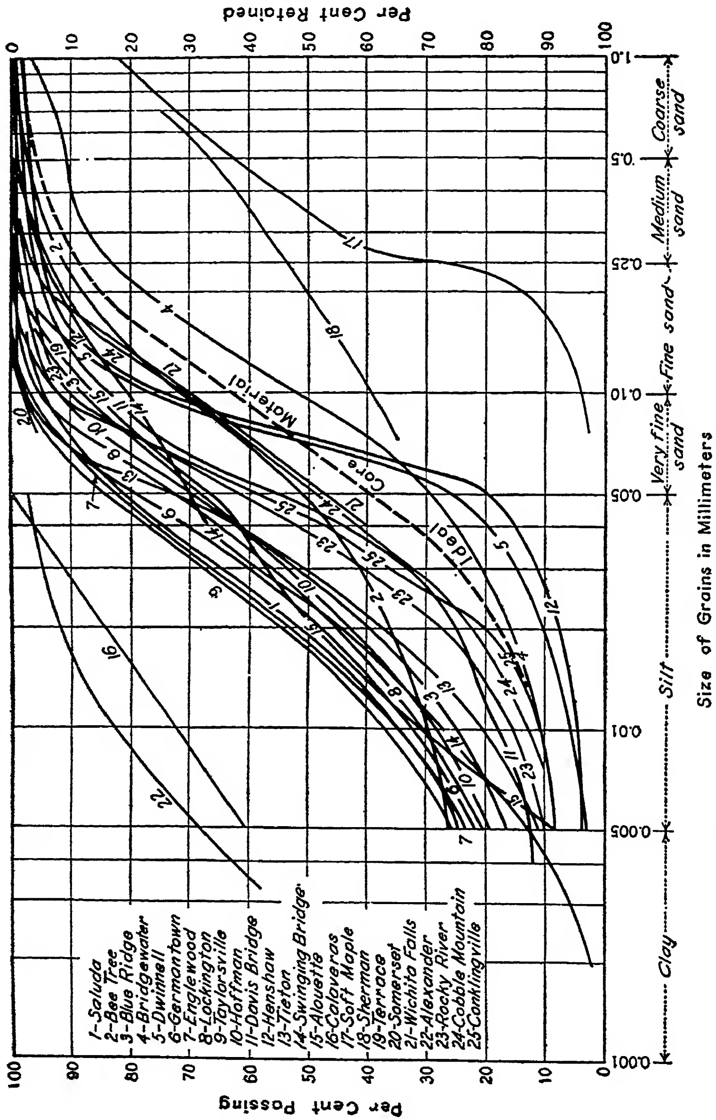

FIG. 29. Mechanical analyses of core material of hydraulic and semi-hydraulic fill dams. (See also Fig. 30.)

An ideal borrow pit material for a hydraulic fill dam might have 15 to 30 per cent of fine material (silt and fine sand) from 0.005 mm to 0.15 mm, most of which will be for the core and the rest of the material grading upward from fine sand 0.15 mm to cobbles 6 in. or more in diameter.

Between these extremes there is a considerable range of materials which may be successfully utilized for hydraulic fill dams. In Fig. 28 are given typical mechanical analyses curves for the borrow pit material of several hydraulic fill dams.[10] Figs. 29 and 30 give analyses of cores and Fig. 31 of shells.

Colloidal material (Art. 3, Chapter 16) in a core should be avoided; even if it is present in the borrow pit, it can be wasted from the core pool. The coarse clays and silts will make satisfactory cores.

It was found by the late Allen Hazen [11] and others that when the effective size of the core material is not less than 0.01 mm the core consolidates in a very satisfactory manner, but that if it contains a high percentage of very fine colloidal material, it does not consolidate perceptibly during construction. Thus it is sometimes desirable to waste the finest of the fines, but if all the fines are extremely fine, it will be necessary to use them and to consider the resulting increased core pressure in the design.

36. Desirability of Narrow Cores. Accidents have occurred in the past owing to the fact that shells were made too narrow to resist the semiliquid pressure of

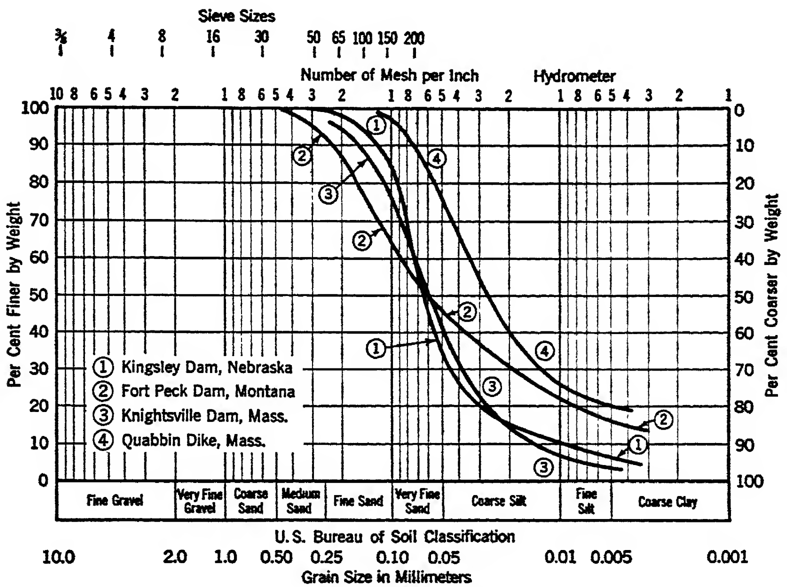

FIG. 30. Typical mechanical analyses of cores of recently constructed hydraulic fill dams.

the core. In general it is desirable to have the hydraulic fill core as narrow as practicable down to a core slope of 3 on 1 or 4 on 1. Shell material, i.e., material

[10] ALBERT S. CRANE, Fig. 155, in Creager and Justin, *Hydro-Electric Handbook*, 1927. E. W. LANE, in *Eng. News-Record*, V. 105, p. 965. *Earth Dam Projects*, 1932, Fig. 58. Personal records of the authors, 1942.

[11] ALLEN HAZEN, "Core Studies in Hydraulic Fill Dams," *Trans. Am. Soc. Civil Engrs.* Vol. 85, p. 1204.

outside the core, has a much greater shear strength than the core material. Therefore, other things being equal, a higher factor of safety against the internal pressure of the core will result by increasing the amount of shell material, decreasing the amount of core material, and steepening the slope of the core. (See also Art. 21, Chapter 18.)

In some cases in the past too much emphasis has been placed on getting the core as tight as possible. Occasionally the effort to obtain as tight a core as possible has led to increased width of core and a decrease in the stability of the shell, with the result that a slide occurred during construction.

An important point about the relationship of the core and shell of a hydraulic fill dam is that the shell should be tremendously more pervious than the core. If the shell is 100 times as pervious as the core, the necessary requirements of stability and relative permeability will have been met. Many shells are at least 1000 times as pervious as the cores. The seepage through the core is practically never of appreciable economic importance even in hydraulic fill dams used for water supply.

In view of the above, if only 10 or 15 ft of the central portion of a hydraulic fill dam is true core, this is generally enough to insure adequate watertightness. The greater width of the core is just to make sure that we will obtain a portion of the core at the center which is free from sand intrusions.

The minimum width of the core should be not less than 20 ft because it is usually impracticable to construct a core narrower than this and be sure to avoid getting sand lenses through it. For this reason it is usual to require that the top 25 or 30 ft of the hydraulic fill dam or at least its core be constructed by rolled fill methods.

37. Intrusion of Sand Lenses Into Core. The intrusion of sand lenses into the impervious core is objectionable only to the extent that it results in the increase of seepage through the dam to an amount which would be sufficient to be of economic importance or which is sufficient to affect the safety of the structure. Sand lenses projecting into the core to some extent are present in every hydraulic fill dam and cannot be entirely avoided, but they can be minimized by care in construction. In fact, lenses of sand projecting a little way into the core are a positive benefit as they hasten the consolidation of the core and improve stability. Sand lenses are a frequent source of argument between the engineers and contractors and their removal is often a matter of great expense.

The usual specifications for hydraulic fill dams are in serious need of revision in this and other respects in the interests of economy and safety. Many specifications require the removal of all sand lenses within the limits of the core, thus materially and unnecessarily increasing the cost of the work.

Specifications should, it is believed, permit sand lenses, regardless of thickness, to project into the core from either side 25 per cent of the theoretical core width (see Fig. 33) at any given elevation. Any sand lenses which project into the core farther than this should be removed or thoroughly broken up. In other words, the central 50 per cent of the theoretical core should be maintained of impervious core material free from sand lenses.

38. Removal of Sand Lenses. Any sand lenses that violate the above criterion can be broken up by water jets which are run down through them at close spacing. As the disturbed material settles back again after jetting there is a strong tendency for it to fall back in stratifications. However, if the jetting is done persistently enough, the continuity of the lenses can be broken up. Engineers have also tried various forms of rakes and plows pulled through the core by

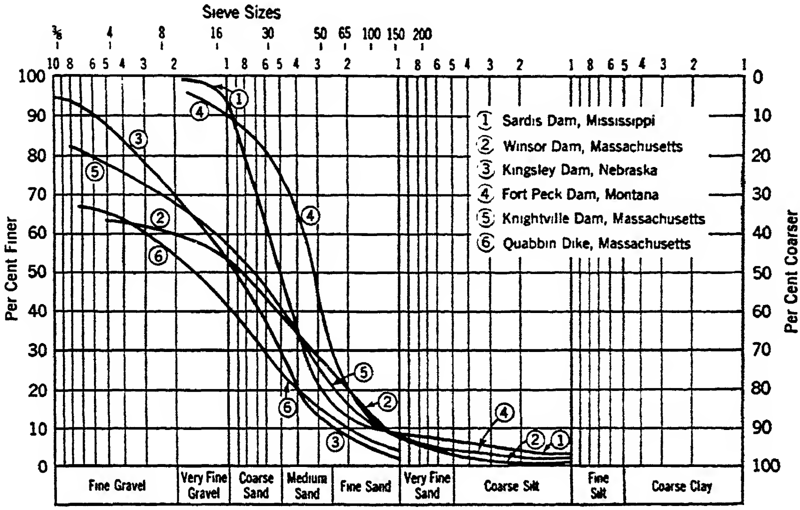

FIG. 31. Typical analyses of shell material of recently constructed hydraulic fill dams.

cables. All of these methods—jets, plows, and rakes—obtain only indifferent success.

By far the most effective method of breaking up and removing sand lenses in the core of a hydraulic fill dam is by the use of a "baby dredge." In its simplest form this baby dredge may not cost more than \$3000 or \$4000 and may consist of a flat-bottom barge propelled by a paddle wheel at the rear operating by chain or belt from a second-hand automobile engine. Another second-hand automobile engine operates a 6-in. centrifugal pump whose suction is located at the end of a short length of rubber suction pipe. No cutter is utilized, but the suction is of the sand-sucker type. The 6-in. dredge pump discharges to a skirt sticking over one side of the barge to spread out the discharge into a thin sheet for the length of the barge. The result of this is that the sand lens is cut through, the hole is refilled with core material which squeezes in, and the sand is dispersed over a considerable area of the core.

Several of the dredges up to 8 in. in size were utilized in the core pool of the Kingsley Dam and proved very effective. In a dam of any ordinary size, say 3 to 6 million cu yd, only one such dredge would be required. In Fig. 32 is shown one of the baby dredges used in the core pool of the Kingsley Dam on the North Platte River, near Ogallala, Nebr. (28,000,000 cu yd of hydraulic fill). In this

case the dredge pulled itself along by means of ropes anchored at various points on the beaches.

39. Lenses of Core Material in Shell. The prevention of tongues or lenses of core material projecting outward material distances into the shells is at least as important as the prevention of tongues of shell material projecting into the core.

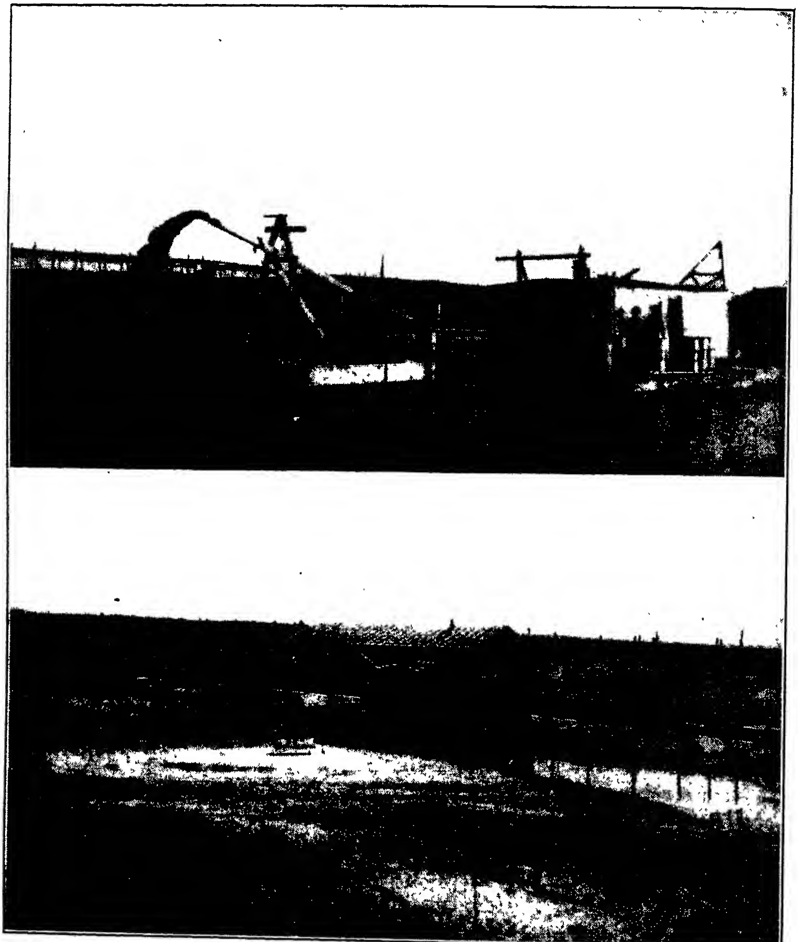

FIG. 32. Baby dredge (6 in) removing sand lenses, Kingsley Dam. (*Courtesy George P. Leonard, Construction Manager, Minneapolis Dredging Co.–Martin Wunderlich Co.*)

Unfortunately many of the specifications have a lot to say about the latter, but they say nothing at all about the former. As a practical matter there is much more hazard to the safety of the structure in a thick lens of core material penetrating a large part of the distance through the shell than there is in a lens of shell material penetrating part way through the core.

It is recommended that at a point outside of the theoretical core line by a distance equal to 25 per cent of the theoretical core thickness at that elevation any lenses of core material shall not be more than 2 in. thick. At a point outside of the theoretical core line by a distance equal to 50 per cent of the core width, there should be no lenses of core material at all. If lenses of core material are found which violate the above criteria, they should be removed by drag line or some other method.

40. Criteria for Lenses in Core and Shell. In Fig. 33 the recommended criteria for lenses of core in shell and lenses of shell material in core which have been discussed above are presented graphically. The advantages of these criteria are:

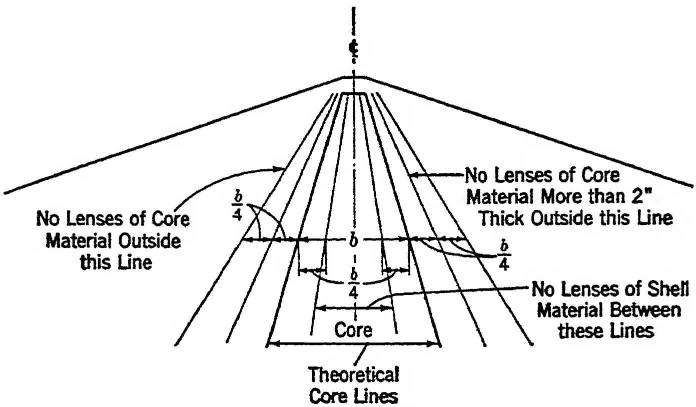

FIG. 33. Recommended requirements for lenses, hydraulic fill dams.

(1) The intended degree of stability will be obtained. (2) The core will be sufficiently impervious. (3) The requirements are definite and a contractor knows just what the requirements are. (4) The requirements are sufficiently broad so that it is practicable to meet them. With careful workmanship there would not be any excessive cost for removal of lenses. (5) The requirements permit a construction organization to vary the actual width of the core within considerable limits so that they may use within limits the available percentage of core material in the borrow pits without the necessity for either importing additional fines or making provision for wasting them.

41. Fort Peck Dam. The Fort Peck earth dam on the Missouri River in Northeastern Montana has a maximum height of 242 ft. The length of the main dam is approximately 10,600 ft. The total length, including the dike on the left or east abutment, is approximately 4 miles. The total contents is about 124 million cu yd, making it by far the largest earth dam so far built (1942).

The dam was constructed by the full hydraulic fill method, utilizing large dredges operating in borrow pits in the floor of the valley. In some cases, in order to obtain suitable material, it was necessary to utilize borrow pits over 5 miles from the dam and pump in several stages to reach the core pool. Construction operations were started with one dredge on October 30, 1934. The

dam was fully completed in 1941. The average season for dredging was 7 months, and an average day's work of four dredges was 124,600 cu yd. A very considerable percentage of fines had to be wasted.

In Fig. 34 is shown a cross-section of the Fort Peck Dam as originally designed and constructed. In this design the relative steepness of the upstream face (average 1 on 4) as compared with the downstream face (1 on 8.5) is notable,

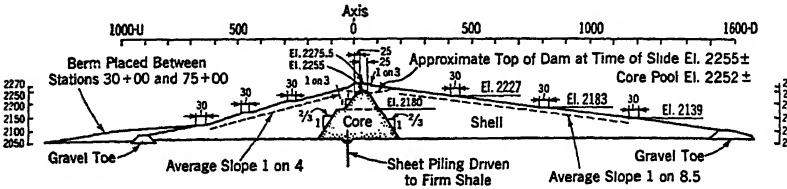

Fig. 34. Fort Peck Dam as originally constructed to elevation 2, 255±.

and also the relative flatness of the core slopes as compared with the core slopes used in most of other recent hydraulic fill dams.

Foundation Conditions. A geologic cross-section of this site on the axis is shown in Fig. 35. The maximum depth of the foundation from the base of the dam to firm shale rock is about 164 ft. The steel sheet piling cutoff penetrated into the weathered shale and in some cases into the firm shale. In general the steel sheet piling extended approximately 20 ft above the original ground surface in order to form a good bond with the impervious core of the dam, resulting in a maximum length of steel sheet piling of about 170 ft.

From Fig. 35 it appears that the Missouri River at this point has been a re-grading stream in past ages. At one time the river flowed over the foundation stratum which is Bear Paw shale, a dark bluish-gray clayey shale of marine origin. When unweathered it is a relatively firm shale, but on exposure to air it slacks and weathers to a fat clay within a short time. Some individual boulders go to pieces within a few days. This process of weathering went on before and during the period when the river valley was being filled up and resulted in the zone of weathered and disintegrated shale along the shale contact. Also erosion from the hillsides washed disintegrated Bear Paw shale onto the valley floor to help form the extensive lenses of fat and lean clays, sometimes intermixed with sands and gravels from a greater distance. Tests on the disintegrated and weathered shale and the weathered bentonite seams which it contained showed the material to have a very low shear strength.

Core of the Dam. The core of the Fort Peck Dam was not as fine as one would expect from an examination of the borrow material, but this is because the core pool pumps wasted back into the river a very large part of the finest material received. As indicated in Fig. 36, there was about 23 per cent finer than 0.01 mm and about 35 per cent coarser than 0.1 mm (fine sand). It might be called a sandy silt. It was quite stable, as indicated by the fact that some of it stood up almost vertically immediately after the slide. At depths in the core pool of 50 ft or more, shear tests indicate a value of tan ϕ of at least 0.6 (angle of internal friction 31°) and only a very slight cohesion.

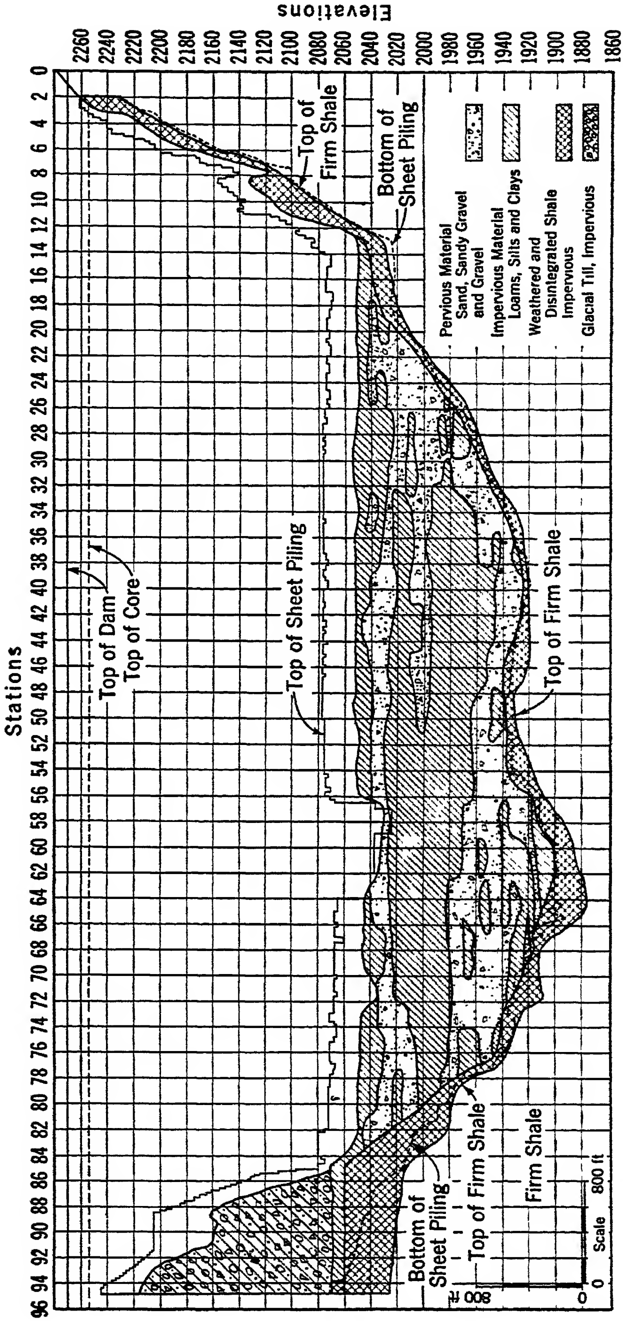

FIG. 35. Cross-section of Fort Peck dam site, looking downstream.

The Shell of the Dam. Tests showed the angle of internal friction of the shell to be not less than 36°, and there was no cohesion. The shell material, as indicated in Fig. 36, had an effective size of about 0.13 mm and a uniformity coefficient of about (0.38/0.13) = 3.0. Many of the analyses curves for the shell indicated uniformity coefficients somewhat higher. Nonuniformity is greatly to be desired in the shell of a hydraulic fill dam, and the higher the uniformity coefficient the more nonuniform the material. Also the steeper the mechanical

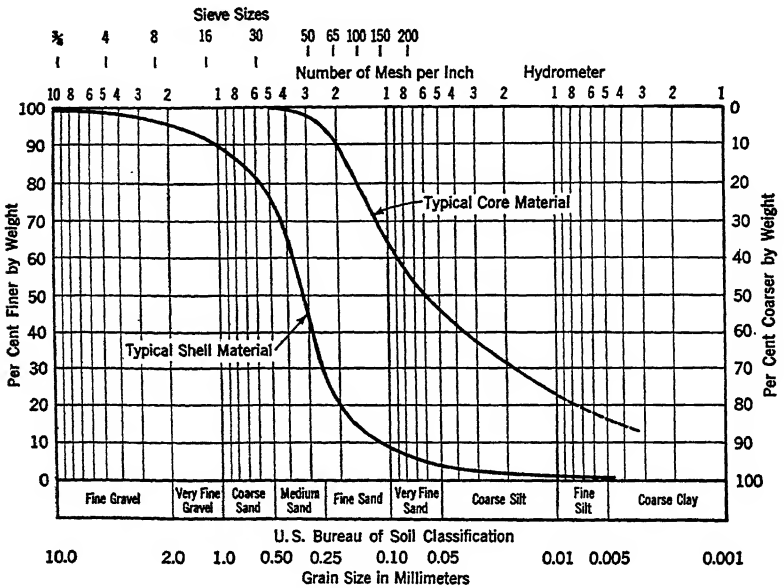

Fig. 36. Mechanical analyses of hydraulic core and shell material of Fort Peck Dam, Montana.

analysis curve the more uniform the material. Thus the shell of the Fort Peck Dam is somewhat finer and more uniform than is desirable to an extent which makes the critical density a matter of interest.

When shell material is fine and the uniformity coefficient is relatively low, there is some danger that the shell material will be less dense than critical density. (See Art. 13, Chapter 16.) Many tests indicated the shell material to be as dense or denser than the "critical density" of the material.[12]

As far as the relative permeability of the shell was concerned it was excellent, as the shell was over 1000 times as pervious as the core. Using Fig. 36 and Table 2 of Chapter 16, the ratio of permeability of shell to core is found to be approximately 1500.

The Slide. On September 22, 1938, when the hydraulic fill had reached elevation 2255± (20 ft from top of the dam) and the core pool had reached 2252± a

[12] T. A. Middlebrooks, "Fort Peck Slide," *Trans. Am. Soc. Civil Engrs.*, 1942.

slide occurred in the upstream portion of the dam near the right abutment. At the time the water in the reservoir stood at elevation 2117.5, having been gradually drawn down from elevation 2136 during the previous 2 months.

The total length of time consumed in the slide was about 10 min. Approximately 5,000,000 cu yd or less than 5 per cent of the material in the dam moved. There were about 180 men in the slide area at the time it started and 8 of them lost their lives.

Then portions of the upstream shell nearest the pool began to slide into the sinking core pool. In a lesser degree, similar cracks and sliding and slumping were taking place on the upstream portion of the downstream beach. Simultaneously with these developments, the main mass of the upstream shell, almost intact, was moving out into the reservoir, in a swing similar to that of a gate hinged at the east (right) abutment.[13]

Investigation of Slide. The Chief of Engineers, General Julian L. Schley, at once appointed a board of consulting engineers to investigate the causes of the slide and to determine a suitable and safe scheme of reconstruction. A comprehensive investigation of the slide and of all pertinent conditions was at once undertaken. A large number of exploratory drillholes 3 to 6 in. in diameter were put down in the affected area. Undisturbed samples of the material in the slide area were obtained from holes 12 to 36 in. in diameter. Exploratory shafts and tunnels were put into the shale of the right abutment. At a number of points in the slide area refrigeration methods were used to freeze the moved material all the way down to the shale. Thirty-six-inch calyx borings were made through this frozen material and the undisturbed frozen cores were preserved for study by the engineers and the consultants. All in all the investigation was probably the most extensive and comprehensive of its kind.

General Results of Investigation. The explorations and the tests on the undisturbed samples obtained showed that (1) the upstream shell had sufficient strength against any pressure which might have been exerted by the core, (2) the upstream shell of the dam had its stratifications preserved in the slide area, (3) the weathered shale and weathered bentonite seams showed shear strength which resulted in a factor of safety of less than unity against shear through the foundation in this section, (4) cumulative evidence of foundation shear through the weathered shale with its weathered bentonite seams was found, (5) there was clear evidence of excess hydrostatic pressure in the firm shale and weathered shale in some cases amounting to 100 ft above the line of saturation in the dam, thus decreasing the effective weight of the embankment.

The consulting board concluded that the slide "was due to the fact that the shearing resistance of the weathered shale and bentonite seams in the foundation was insufficient to withstand the shearing force to which the foundation was subjected. The extent to which the slide progressed upstream may have been due, in some degree, to a partial liquefaction of the material in the slide." [14]

[13] From "Report of the Slide of a Portion of the Upstream Face of Fort Peck Dam," July 1939, p. 4, Engineer Corps, U. S. Army.

[14] From "Report of the Board of Consulting Engineers on the Fort Peck Slide."

It should be noted that the downstream portion of the dam, the slope of which was twice as flat as the upstream slope, did not move at all. Another notable fact, as shown by Fig. 34, is that from Station 30 to Station 75, there was an upstream berm at elevation 2112 but that this berm was lacking in that portion of the dam where the slide occurred. Fair inferences from the observed simple facts were: (1) Strength of foundation was insufficient to support relatively concentrated load of dam with 1 on 4 slope but would successfully support dam with

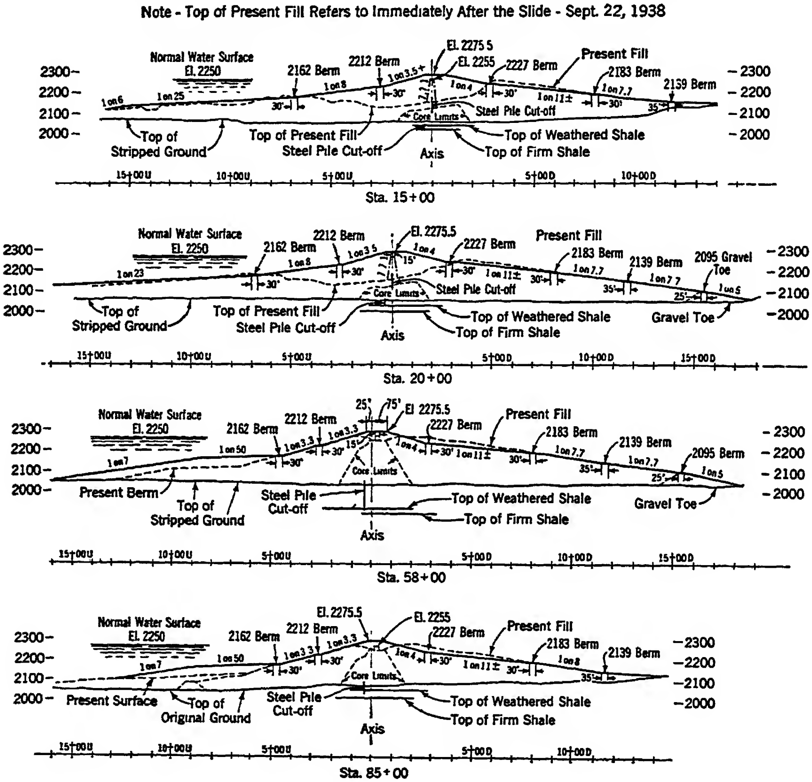

FIG. 37. Cross-sections of Fort Peck Dam, Montana, as reconstructed. *(Adopted from Fig. 25, "Report on the Slide of a Portion of the Upstream Face of Fort Peck Dam," Corps of Engineers, U. S. Army.)*

1 on 8 slope; (2) the factor of safety of a portion of the upstream face beyond Station 30, which did not fail, was probably slim because the only difference between the portion which did not move and that which failed was the relatively slim berm at elevation 2112.

These inferences were borne out by the investigation, but there were, of course, many complicating factors.

In accordance with the recommendation of the board, the dam as reconstructed as shown by cross-sections in Fig. 37. The additional core required was made

very narrow and was placed and compacted by rolled fill methods. As indicated in Fig. 37, some of the core was sheared off in the slide, leaving shell material overlapping the remaining core and here the old core and the new core were connected by means of steel sheet piling as shown in Fig. 37.

42. Kingsley Dam, Nebraska. The Kingsley Dam is located near Ogallala, Nebr., on the North Platte River. The maximum height of the main dam is 165 ft. The length of the main dam is approximately 13,100 ft and the dike continues 3000 ft farther on the left abutment. The dam contains approximately 28,000,000 cu yd of embankment, making it the second largest earth dam in the world (1942). Like the largest earth dam, Fort Peck, it was constructed by full hydraulic fill methods except for the relatively shallow topping off process which is, in general, necessary with all hydraulic fill dams.

The foundation formation at this point consists of a dense massive thoroughly consolidated silt which, although it cannot properly be termed ledge rock, has an ample shear strength to take care of any load which might come on it. The North Platte River had filled in the original stream valley with sand gravel and some boulders to a depth ranging from 40 to 130 ft. The flood plain of the valley was, in general, covered with 3 ft to 10 ft of silt. Below this point there was very little silt mixed with the sand gravel of the valley fill, which in general was pervious but very stable. The right abutment consisted of the so-called Ogallala formation, a series of strata of sands, gravels, and silts partially cemented by deposits of lime. The left abutment consisted of gently rising deposits of blow sand, which had been found by investigation to have a density somewhat greater than "critical density." As a precautionary measure the blow sand portion of the foundation was thoroughly saturated before and during loading and an additional width of embankment added on the upstream side of the dam over the blow sand area.

As shown in Fig. 38, a steel sheet piling cutoff (Carnegie Illinois Section M 112, $\frac{3}{8}$ in. thick, 23 lb per sq ft) was driven into the brule formation, which was massive and impervious. The maximum depth of steel sheet piling was approximately 145 ft. To reach such depths it was necessary to use as a maximum a gang of three jets on each side of the row of steel sheet piles, three of which were generally driven together. The pumping plant for the jets had a capacity 300 gpm under a 250-ft head. Some compressed air was also used at a head of 300 lb per sq in. This water and air under heavy pressure practically liquefied the foundation, and with very few exceptions the piles were put down and seated in the brule with little trouble.

At the steep right abutment the steel sheet pile cutoff ties into a concrete core wall let into the Ogallala formation by means of stoping methods for a distance of about 500 ft into the abutment from the point where normal reservoir surface would intersect the abutment. The bottom of this concrete wall was bonded to the brule. The above horizontal distance to which the concrete wall was carried into the abutment was for the purpose of securing a satisfactory percolation ratio through some of the more pervious strata in the loosely cemented Ogallala formations.

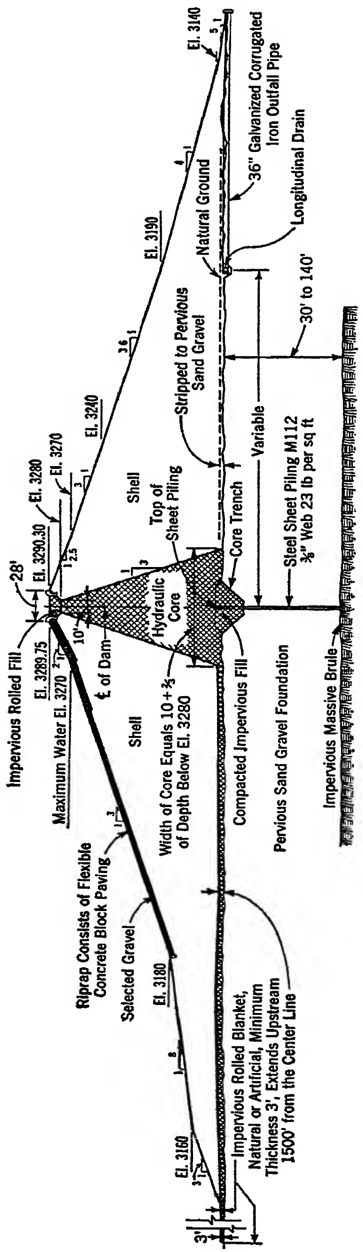

FIG. 38. Kingsley hydraulic fill dam, Ogallala, Nebr.

On the gently sloping left abutment the steel sheet piling cutoff to the Brule was terminated at Station 110, where the original ground surface was elevation 3250 (20 ft below normal headwater). From here to the north end of the dike, the cutoff consisted of a trench filled with compacted impervious material (loess). The depths of the partial cutoff ranged from 40 ft on up to 10 ft at the end of the dam.

In addition to the positive cutoff provided by the steel sheet piling, it will be noted that the natural impervious blanket was preserved and it was also patched where necessary to obtain a continuous blanket.

For excavating and placing the fill a 28-in. dredge was used on the downstream side of the dam and another 28-in. dredge on the upstream side of the dam. The valley fill excavated and pumped from the valley floor was deficient in fines. Consequently, the additional fines were provided by a separate operation. A loess soil which is both stable and impervious was excavated in the dry from borrow pits on the right abutment, dumped into a hog box and hydraulicked to the core pool. The delivery pipes for this purpose were provided with windows and were supported on pontoons in the center of the core pool. The core material in the dam had, in general, an effective size of 0.01 mm, which, though coarser than that in some hydraulic fill dams, is nevertheless relatively tight.

A typical mechanical analysis for the shell material is shown in Fig. 31 and a similar curve for the typical core is shown in Fig. 30.

43. Bibliography:

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2. "The Talla Dam," *Eng. News-Record*, Vol. 51, p. 21.

3. *Design and Construction of Conchas Dam*, U. S. Engineer Office, Caddoa, Colo.

4. "Core Studies in the Hydraulic Fill Dams of the Miami Conservancy District," *Trans. Am. Soc. Civil Engrs.*, Vol. 85, 1922, p. 1181.

5. E. W. LANE, "Materials in Earth Dams," *Eng. News-Record*, Vol. 105, p. 961.

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7. T. A. MIDDLEBROOKS, "Fort Peck Slide," *Trans. Am. Soc. Civil Engrs.*, Vol. 107, p. 723. (1942.)

8. Report of the Slide of a Portion of the Upstream Face of Fort Peck Dam, July 1939, Engineer Corps, U. S. Army.

9. JOEL D. JUSTIN, "Design of Earth Dams," *Trans. Am. Soc. Civil Engrs.*, Vol. 87, 1924, p. 1.

10. JOEL D. JUSTIN, *Earth Dam Projects*, John Wiley & Sons, Inc., New York, 1932.

11. CREAGER and JUSTIN, *Hydro-Electric Handbook*, Chapter 13, John Wiley & Sons Inc., New York, 1927.

12. Also note the many references to articles on representative earth dams in Table 2

CHAPTER 20

ROCK-FILL DAMS [1]

1. General. Rock-fill dams are used rather extensively in remote locations where the cost of cement for a concrete dam would be high, where suitable materials for earth dams are not available, and where suitable rock can be quarried at or near the dam site. Attention is also called to the fact that a foundation might be acceptable for a rock-fill dam and not acceptable for a concrete dam. A number of these dams have been built to more than 200 ft in height and one dam, Salt Springs (Fig. 10), has been built to 328 ft. Such dams must of necessity include an impervious element, which may be either an internal core wall or an upstream impervious facing, the latter type being now considered superior.

In modern practice the rock-fill dam has three fundamental parts: (1) the dumped rock fill, (2) an upstream rubble cushion of laid-up stone bonding into the dumped rock, and (3) an upstream impervious facing resting on the rubble cushion. In some dams the rubble cushion is replaced by a graded filter and the impervious diaphragm on the upstream face is replaced by a relatively heavy earth fill. Such dams, however, are usually referred to as "composite dams" and are discussed in Art. 11 of this chapter.

2. Foundation and Cutoff Wall. The essential condition with regard to the foundation for a rock-fill dam is that it shall not be subject to material settlement or to erosion from such seepage as may pass through or under it. To prevent such seepage, a concrete cutoff wall at the bottom of the facing is usually essential. It should be bonded into firm ledge rock or other suitable impervious material. If the foundation is anything other than sound ledge rock, the possibility of a foundation blowout must be carefully investigated and guarded against. This concrete cutoff wall should extend across and up the sides of the canyon. Grouting may be required in the ledge rock below the cutoff in order to seal the dam against seepage under the structure.

The connection between the impervious diaphragm on the upstream face of the dam and the concrete cutoff wall should be flexible so that a material amount of movement in the slab may take place without causing a rupture which would produce extensive leakage. The problem is particularly difficult at the junction of cutoff walls up the sides of the canyon with the flexible concrete facing, and the design at this point should permit a large amount of movement without danger of fracturing the concrete facing. The cutoff wall should be able to take any thrust which may be transmitted to it from the impervious diaphragm on the upstream face. There is usually little or no settlement at the cutoff, but a short distance away from it the settlement may be considerable.

[1] A large part of the data for this chapter was compiled by the late Carl Ashley.

3. Cross-Section. Modern rock-fill dams are usually built on a ratio of base to height ranging between 2.5 and 3.0. The downstream slope is usually made the natural slope of rock dumped from cars or trucks or about 1 on 1.3 to 1 on 1.4. If the slope is steeper or flatter than this it usually requires additional handling. The upstream slope ranges from the natural dumped rock slope to about 1 on ¾. Slopes as steep as 1 on ½ have been used, as in the case of the Beaver Park Dam

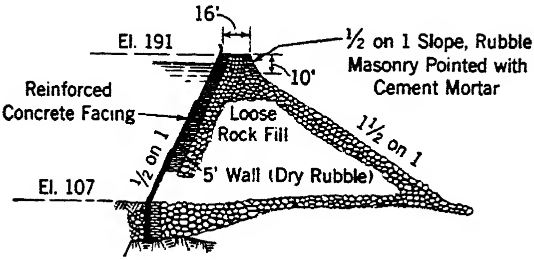

FIG. 1. Section through Beaver Park Dam, Colorado. (*Eng. News, Vol. 73, p. 660.*)

in Colorado (Fig. 1) and Relief Dam on Stanislaus River, California (Fig. 2), but later practice is to build the upstream slope with the natural dumped rock slope of about 1 on 1.3, as in Fig. 10.

The upstream face is usually made concave along the water slope to prevent buckling of the facing when settling occurs. Dams exceeding 100 ft in height

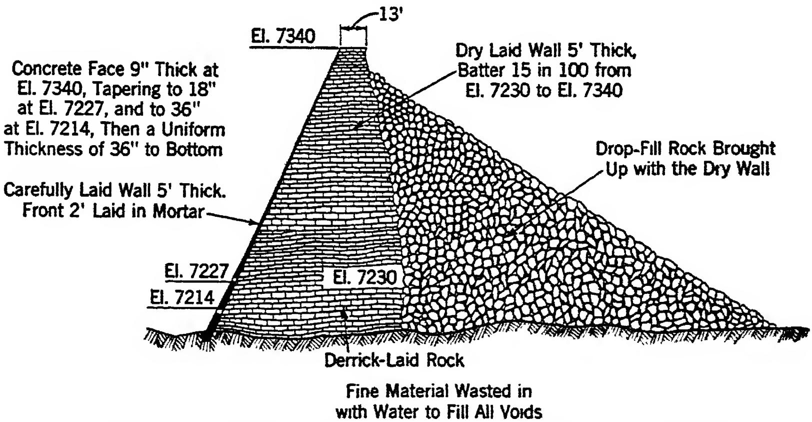

FIG. 2. Relief dam on Stanislaus River, California. (*Trans. Am. Soc. Civil Engrs., Vol. 75 (1912), p. 55.*)

should have a crest width of not less than 15 ft and even low dams should not have a crest width of less than 10 ft. The water pressure on a rock-fill dam is resisted only by the weight of the rock. No arching or cantilever action can be considered as aiding stability.

4. Safety Against Sliding. Almost any rock-fill dam with an impervious upstream face which it would be practicable to build on a suitable foundation will

TABLE 1—IMPORTANT

Dam	River	Location	Date Completed	Approx. Volume (cu yd)	Approx. Height Above Foundation (ft)	Crest Width (ft)	Crest Length (ft)	Base Width (ft)
Bowman (First)	S. Yuba	Calif.	1876	55,000	96		425	
North Bowman	S. Yuba	Calif.	1927	307,000	168	15	680	
Fordyce (First)	S. Yuba	Calif.	1873–81		93	5	800	139
Fordyce (Enlarged)	S. Yuba	Calif.	1926	417,000	130	10	1060	335
French Lake	S. Yuba	Calif.	1873		68	6	250	
Walnut Grove	Hassayampa	Ariz.	1888	46,000	110	10	400	140
Castlewood	Cherry Cr.	Colo.	1890		70	8	·600	
Chatsworth Park	Mormon Cr.	Calif.	1896	6,000	41	4	159	
Escondido	San Elijo Cr.	Calif.	1895	37,000	76	10	380	140
Lower Otay	Otay Cr.	Calif.	1897	140,000	135	16	565	280
East Canyon	E. Canyon Cr.	Utah	1899–1902	39,000	93	10	173	289
Bear River (First)	Mokelumne	Calif.	1900	56,000	75	13	748	133
Bear River (Enlarged)	Mokelumne	Calif.	1932	85,000	80	7	755	164
Meadow Lake	Mokelumne	Calif.	1903	46,000	74	12	775	103
Skaguay	Beaver Cr.	Colo.	1901	42,000	70	20	405	148
Sabrina	Bishop Cr.	Calif.	1909	47,000	70	10	1065	150
South Lake	Bishop Cr.	Calif.	1910	75,000	80	10	650	170
Morena	Cottonwood Cr.	Calif.	1912	324,000	167	16	520	389
Relief	Stanislaus	Calif.	1910	137,000	140	13	505	290
Cucharas	Cucharas	Colo.	1911	195,000	125	20	550	
Swift	Birch Cr.	Mont.	1914		165	15	465	478
Strawberry	Stanislaus	Calif.	1916	330,000	150	15	612	382
Beaver Park	Beaver Cr.	Colo.	1914		87	16	370	
Drew's Dam	Drew's Cr.	Oreg.	1915	45,000	65	10	610	170
Dix	Dix	Ky.	1915–25	1,747,000	275	20	1032	712
Bucks	Feather	Calif.	1928	347,000	122	12	1220	320
Bonito	Bonito	N. Mex.	1931	147,000	102	15	440	250
Salt Springs	Mokelumne	Calif.	1931	3,200,000	328	15	1300	905
San Gabriel No. 2	San Gabriel	Calif.	1935	1,200,000	280	18	600	785
Penrose-Rosemont	Beaver Cr.	Colo.	1932		100	22	580	206
Kébir	Kébir	Tunis	1925–32		115	25	1100	
Bakhadda	Haute Mina	Algeria	1928–33	420,000	148	16½	722	354
Ghrib	Chéliff	Algeria	1936	875,000	233	16½	886	480
Bou Hanifia	Hammam	Algeria	1932	1,000,000	180	16½	1510	410
Foum-El-Gueiss	Gueiss	Algeria	1934	170,000	75½	10	820	208
Tepuxtepec	Lerma	Mexico	1929	91,000	126½	15	900	240
Cogoti		Chile	1939		246			
English	Mid-Fork Yuba	Calif.	1856		100		331	185
Nantahala	Nantahala	N. C.	1942		260	30	1040	990 ±

Rock-Fill Dams

Slope: 1 Vertical on—Horizontal		Rubble Thickness		Facing or Core	Thickness of Face or Core		Remarks
Up-stream	Down-stream	Top (ft)	Bottom (ft)		Top (in.)	Bottom (in.)	
1	1	6	18	Timber	3	9	Log crib, 1872; enlarged 1876; dismantled 1926.
$\frac{1}{2}$, $\frac{3}{4}$	1.4	$5\frac{1}{2}$	20	R. conc.	8	12	
1	$\frac{1}{4}$, $\frac{1}{2}$			Timber			Enlarged 1926.
1	1.35	4	6	R. conc.	12	18	Monolithic flexible type face.
$\frac{1}{2}$, 1	$\frac{1}{2}$			Timber			
0.43	0.65, 1	4	14	Timber	6	6	Failed 1890.
0.1	.1	4	6				Upstream earth fill added. Failed 1933.
0.57	0.87	2	2	Conc.	8	16	
$\frac{1}{2}$	1, $1\frac{1}{4}$	5	15	Timber	2–$1\frac{1}{2}$	2–3	2 in. of concrete back of plank.
$1\frac{1}{4}$	$1\frac{1}{4}$	None	None	Steel core	$\frac{1}{4}$	0.34	Steel plate core wall protected by concrete (see Fig. 11), failed 1916.
$\frac{2}{3}$ and vert.	2	None	None	Facing-core	$\frac{1}{4}$	$\frac{3}{8}$	First 68 ft, added 25 ft in 1902.
$\frac{1}{2}$, $\frac{3}{4}$	$\frac{1}{2}$, $\frac{3}{4}$, 1	8	16	Timber	2–$1\frac{1}{2}$		Enlarged in 1932.
$\frac{1}{2}$, $\frac{3}{4}$	$\frac{1}{2}$, 1.35	8	16	Timber	2–$1\frac{1}{2}$		
$\frac{1}{2}$, $\frac{3}{4}$	$\frac{1}{2}$	6	7	R. Gunite	2	4	Original timber face burned 1929.
0.58	1.2	None	None	Steel	$\frac{1}{4}$	$\frac{1}{2}$	
$\frac{3}{4}$	$1\frac{1}{4}$	5	6	Timber	5	9	Facing renewed 1929.
$\frac{3}{4}$	$1\frac{1}{4}$	5	6	Timber	5	9	Facing renewed 1930.
$\frac{1}{2}$, 0.9	1.5	16	50	Masonry	36	72	Top raised 5 ft in 1917, 10 ft in 1923; conc. blanket for 42 ft above stream bed.
$\frac{1}{2}$	1.5	13	108	R. conc.	12	36	
1	1.5	21	21	R. conc.	18	18–24	
1	$1\frac{1}{4}$, 1.5	4	4	R. conc.	6	24	2 in. sub-base under facing.
1, 1.2	1.35	4	18	R. conc.	9	15	3 in. sub-base under facing.
$\frac{1}{2}$	$\frac{1}{2}$, 1.5	5	5	R. conc.	12	24	Monolithic rigid facing.
$\frac{1}{2}$	1.5	5	16	Timber	$4\frac{3}{4}$	$4\frac{3}{4}$	Resurfaced 1930, two $1\frac{3}{4}$ in. layers.
1, 1.2	1, 1.4	4	14	R. conc.	8	18	Forms left on lower 160 ft.
1.4	1.5	3	7	R. conc.	12	19	Monolithic rigid facing.
1.17	1.4	6	13	R. conc.	8	12	Coated with bituminous material.
1.3 av.	1.4	15	15	R. conc.	12	38	Monolithic flexible type face.
1.2, 1.3, 1.35	1.5	6	15	Timber	3–2	3–2	2-layer conc. facing failed by settling.
$\frac{1}{2}$	1.4	5	11	Steel	$\frac{1}{4}$	$\frac{3}{8}$	About $\frac{1}{3}$ of rubble is set in mortar.
1, 1.5	1, 1.5			Hollow core	6.5	30	Core slid and cracked. Slopes flattened in 1930.
0.86, 1	$1\frac{1}{4}$	8	20	R. conc.	12 & 16	12 & 16	Lower layer 12 in., top layer 16 in.
0.7, 1	$1\frac{1}{4}$	10	33	Bit. conc.	2–$2\frac{3}{8}$	2–$2\frac{3}{8}$	Porous conc. under and over bituminous conc.
0.8, 1	$1\frac{1}{3}$	$11\frac{1}{2}$	$11\frac{1}{2}$	Bit. conc.			
1	$1\frac{1}{4}$	8	13	R. conc.	8	14	One layer on cyclopean masonry.
0.7	0.7, 1.1	6	6	R. conc.	20	20	Rubble in conc. slabs; two layers of facing interlocking.
1.6	1.8	Gravel	Gravel	R. conc.	10	16	
0.87	0.60	None	None	Planked timber crib			First built 1856 to 79 ft, then widened by 44 deg. upstream dry wall and raised 21 ft. Went out 1883 supposedly by being blown up. Earliest high rock-fill dam in U. S.
1.4, 2.5	1.4	None	None	Earth core	180	324	Core slope 1 on 1.42.

TABLE 2

IMPORTANT COMPOSITE ROCK-FILL AND EARTH DAMS

Dam	River	Location	Date Completed	Approx. Height Above Foundation (ft)	Crest Width (ft)	Crest Length (ft)	Base Width (ft)	Slope: 1 Vert. on—Hor.			Core	Remarks
								Up-stream Earth	Up-stream Rock-fill	Down-stream Rock-fill		
McMillan	Pecos	N. Mex.	1893	57	25	2114		2, 3.3	$\frac{1}{2}$	1.5	None	Includes two dikes.
Milner (3 dams)	Snake	Idaho	1905	56, 81, 86	20	1235		4	$\frac{3}{4}$	1.5	Timber	Timber core backed by dry wall.
Lake Avalon	Pecos	N. Mex.	1893–1907	58	34	1025		6′ vert. & 2, $3\frac{1}{2}$	$\frac{1}{2}$	1.5	R. conc.	Failed in 1893 and 1904. Widened and conc. core wall added.
Zuni	Zuni	N. Mex.	1907	70	20	720	296	3	$\frac{1}{2}$	$1\frac{1}{4}$	None	Failed in 1909 but was repaired.
Minidoka	Snake	Idaho	1909	60	25	625	300	3	1	$1\frac{1}{4}$	Conc.	
San Gabriel No. 1	San Gabriel	Calif.	1933–37	380	40	1570		3 (av.)		−3 (av.)	None	
Inland	Little Warrior	Ala.	1938	190	30	1100		3	1.3	1.3, 2	None	
Pleasant Valley	Fish Cr.	Utah	1927	63		450		3	$\frac{3}{4}$	1.5	None	
Glenville	Tuckasegee River	N. C.	1941	165	30	900	840	$\left\{\begin{array}{l}2.5\\3\end{array}\right.$	$\left\{\begin{array}{l}2.5\\3\\3.5\end{array}\right.$	2	Earth	

necessarily have a relatively high factor of safety against sliding because of the large mass involved. Thus J. D. Galloway (1) [2] points out that "On the assumption of a dam 200 ft high with a crest 15 ft wide, a downstream slope of 1 on 1.4 and a unit weight of 100 lb per cu ft for the loose rock and with the water load, ratios of height to base of 1 : 2.25, 1 : 2.5, and 1 : 3 would have sliding factors (ratios of weight of rock to water pressure) of 4.50, 5.14 and 6.45, respectively. These ratios are practically constant for all heights of dams." Nevertheless the adequacy of the foundation should in all cases be investigated.

5. Main Rock Fill. The rock fill in the main part of the dam must be of sound rock which will not readily disintegrate, split, or crush. Thus, shales which slake in the presence of air are dangerous and should be rigidly excluded.

FIG. 3. Construction photograph, Dix River Dam, March 9, 1925. (*Courtesy L. F. Harza, Consulting Engineer.*)

Rock which, when shot, shatters into very small pieces with a high percentage of chips and dust is unsuitable.

Methods of placing rock fill vary with height of canyon walls and location of quarries. At sites with high canyon walls it is often possible to drive powder drifts into the walls in such a way that a considerable portion of the rock fill required may be shot down to, or near, its final position in the dam. At the Morena rock-fill dam (Fig. 5) for the water supply of San Diego, Calif., 180,000 tons of rock was loosened by a single blast in a convenient location under the cableway, and at the Dix River rock-fill dam (Figs. 3 and 6) in Kentucky 159,000 cu yd was shot down by two powder drifts to a position within the dam area

[2] Numbers in parentheses refer to Bibliography, Art. 14.

proper. However, the rehandling and grading of the rock, which is often necessary after such blasts, has led many engineers to question the over-all economy of the procedure except in unusual situations.

The rock fill must usually be transported from the quarry by dump trucks, railroad, derricks, cranes, or cableway. With rail transportation side-dump cars are often used, working out on a circular track from one or both abutments. Where the width of dam is too narrow for such a circular dump track a timber trestle along the line of the dam is employed to start the fill, which is widened

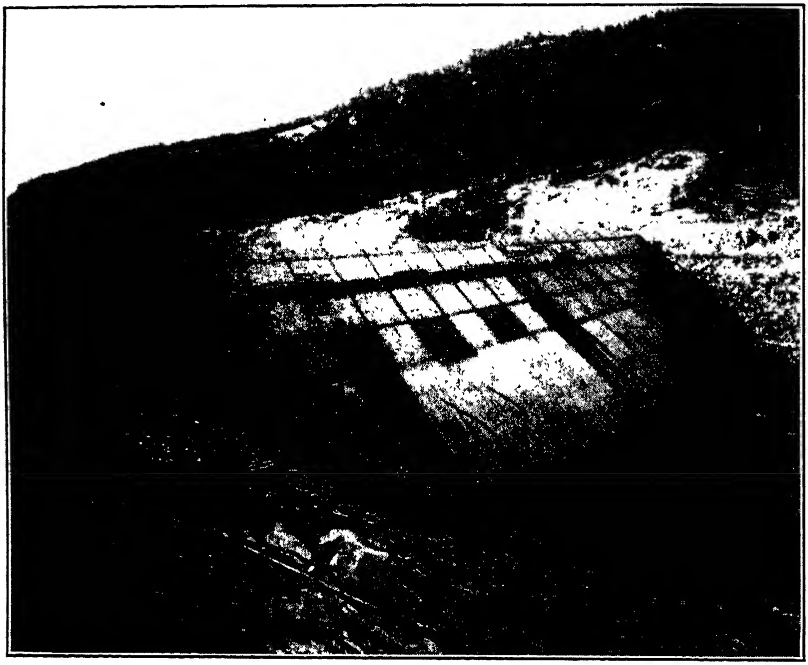

FIG. 4. Salt Spring Dam, California. Reinforced concrete upstream face under construction. (*Courtesy Pacific Gas & Electric Co., San Francisco.*)

by side dumping after reaching trestle level. The trestle timber is left in the fill. Successive lifts are made in this manner. An alternative to the trestle method is to use end-dump cars, working out from the side walls of the canyon, to form a rock-fill bank from which side-dumping may be more efficiently carried on. End-dump trailers drawn by caterpillar tractors were effectively employed at Salt Springs.

With truck transportation the various lifts may be extended across the valley by both end and side dumping. Derricks, cranes, or cableways are used chiefly when the rock fill is obtainable directly at the abutment ends of the dam. In most recent large rock-fill dam construction railroad cars and trucks have transported the rock fill.

Considerable difference of opinion exists as to the height of drop, or lift, which is advisable. At Salt Springs (Fig. 10), with granite, a maximum drop of 165 ft was used, and the settlement in the dam proper was relatively small. The practical limit would be such that it would not seriously injure the rock or be dangerous to workmen.

The size of the individual rocks may vary greatly. With ordinary dump trucks the maximum size is about 3 tons. Using air dumped railroad cars or very heavy trucks it may average 10 tons, reaching a maximum size in recent dams of about 25 tons. The sizes if placed by derricks, cranes, or cableways depend upon the economical machine capacity. As far as practicable chips and dust should be excluded from the fill. Certainly, 10 per cent should be the top limit for such objectionable material under most conditions.

6. Rubble Backing of Impervious Face. The rubble wall between the main rock fill of the dam proper and the concrete or other facing is built of hand- or derrick-laid rock and acts as a cushion equalizing settlement and stabilizing the upstream facing. It should be carefully laid up like a dry rubble wall with large voids chinked with spalls so that it will form a substantial backing for transmitting and distributing the water load on the impervious face to the main body of the dam as in Fig. 4. Rubble cushion walls have been made of varying thickness and the design of this feature is almost entirely a question of judgment. They have been made 5 ft to 50 ft at the bottom to a minimum of 4 ft at the top.

There is an advantage in using a natural slope such as 1 on 1.3 to 1 on 1.5 on the upstream face. If the upstream face is very steep (say 1 on 0.5) the upstream cushion of hand- or derrick-placed stone should be very thick because in effect we have a dry rubble wall retaining the loose rock fill behind it as shown in Fig. 5. Necessarily in this case the hand- or derrick-placed cushion must be built as the loose rock fill is deposited and before the fill has attained practically any initial settlement. On the other hand, if a natural slope is used on the upstream face, the loose rock fill may be entirely completed in dams of medium height and volume and allowed to attain its initial settlement before the construction of the cushion. In recent high rock-fill dams the building of the cushion is begun after some convenient height, say 75 ft, of main loose rock fill has been reached and then follows up the loose rock dump. This practice is necessary because of the time element in completion of large dams and also in order to keep within a reasonably safe drop height for any additional stone required to form the rubble cushion. (See Figs. 3 and 6.)

When the loose rock dump fill is built in advance to a natural slope, many engineers believe that there is no reason for making the derrick- or hand-placed rubble facing thicker at the bottom than elsewhere, although I. C. Steele, of the Pacific Gas & Electric Co., believes that it should be somewhat thicker at the bottom than at the top. The rubble cushion should have sufficient horizontal width to permit the movement of the caterpillar cranes or the setting up of derricks. A thickness for the rubble of 10 ft normal to the slope is probably the minimum advisable except for very low dams.

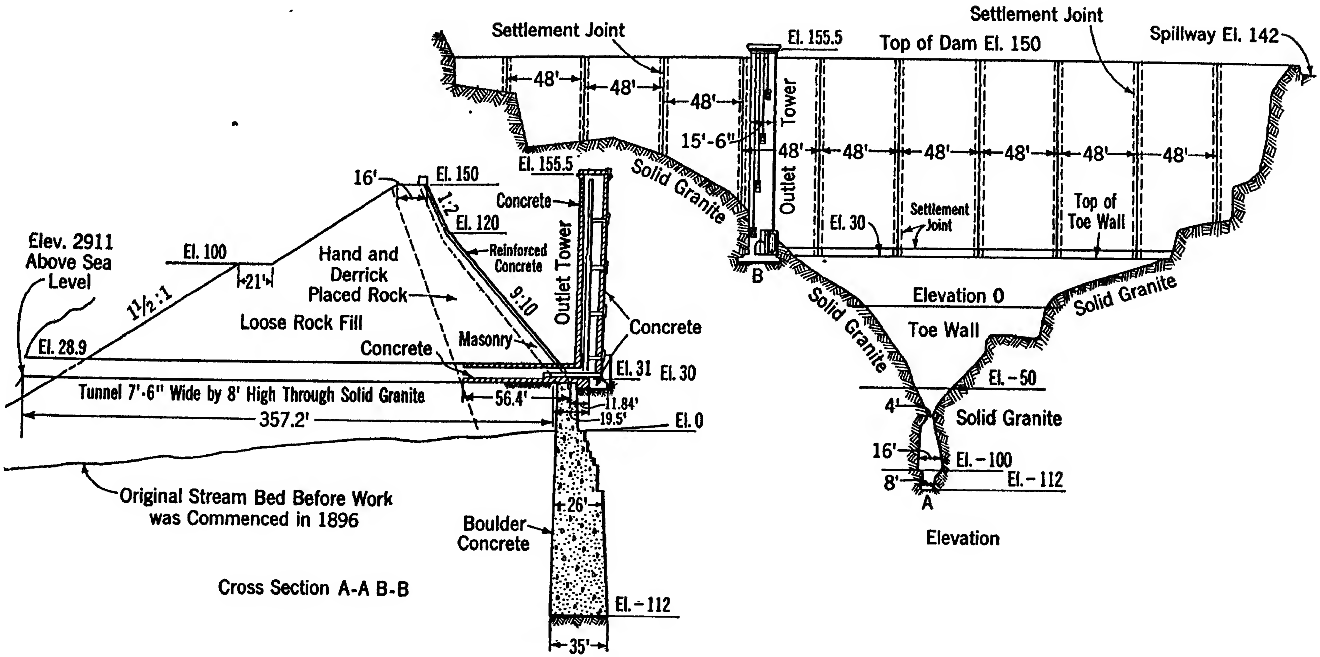

FIG. 5. Morena rock-fill dam, California. (*Trans. Am. Soc. Civil Engrs., Vol. 75 (1912), p. 37.*)

A rock-fill dam with a very steep upstream slope will always contain much less material than one with a natural upstream slope (see Fig. 7), but the quantity of relatively expensive hand- or derrick-placed stone will be very much greater. Both conservatism and economy usually favor the upstream face with natural slope.

Under many conditions the following method of constructing rock-fill dams up to a height of about 150 ft has been found economical. First the foundation, including the canyon walls, is stripped of all loose and erodible material. Then the loose rock fill is started by end dumping from trucks at one or both abutments at the elevation of the top of the dam. The dam is thus built out from the abutment on natural slopes, and the process is continued until the loose rock fill is com-

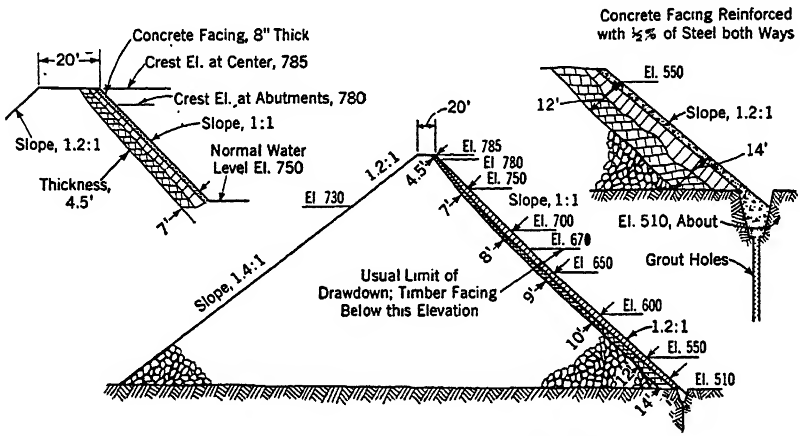

Fig. 6. Dix River rock-fill dam—rubble, concrete, and timber facing on upstream side. (*Eng. News-Record, Vol. 94, p. 1059.*)

pleted. By this method the maintenance of the trucking roads over the loose rock fill is obviated except for the single road at the top of the dam.

As the rock is dumped powerful streams of water are directed against the rock slopes, thus sluicing down out of the fill or into the interstices of the rock fill such fines as may be contained in the rock.

After the loose rock fill is completed near the abutments the construction of the derrick-placed stone cushion is started. If the loose rock fill contains suitable big stones these are taken from the face of the loose rock fill just ahead of the derricks and used in constructing the rubble rock cushion at the upstream face. If the loose rock fill does not contain suitable big stones (500 lb and more) it may be desirable to deliver the stones for the cushion as a separate operation or to use smaller selected stones from the face of the fill to form a hand-placed rubble cushion.

The placing of the reinforced concrete diaphragm which forms the upstream face follows the placing of the rubble cushion as promptly as circumstances will permit.

It will be noted that under this method the total cost of the loose rock fill which forms the body of the dam includes little more than the cost of quarrying, transporting, and dumping the rock.

7. Impervious Upstream Facing. As previously stated, most recent practice in making the rock-fill dam watertight tends to the use of an impervious facing on the upstream slope attached in such a manner as to obtain some degree of flexibility to the cutoff wall at the upstream toe. The facing can be made of wood, steel, or concrete. In a few instances it has been of bituminous concrete.

Timber Upstream Facing. The older dams were usually faced with wood, and its flexibility, which permits the rock dam to settle without significant damage, is quite an advantage. It is used frequently in remote locations where timber is plentiful.

Below the minimum water surface elevation timber facings are practically permanent. Timber was used on the lower portion of the face of the Dix River Dam, Fig. 6. Timber in a rock-fill crib dam with plank face built prior to 1800 at Norristown, Pa., was found to be in practically perfect condition in 1927 below minimum water surface. Above this elevation, the life of a timber facing may be relatively short. Some timber facings have been built of timber impregnated with creosote, as on the Sabrina and South Lake Dams on Bishop Creek, California (1), which were resurfaced with pressure-treated Douglas fir and redwood. Such facings have a relatively long life, even in storage reservoirs where a large portion of the facing is exposed to the air for a material part of the time.

Timber facings are subject to a serious fire hazard at any time when exposed, but this hazard may be largely obviated by the addition of a layer of gunite on wire mesh attached to the timber face or by an efficient patrol system. Timber

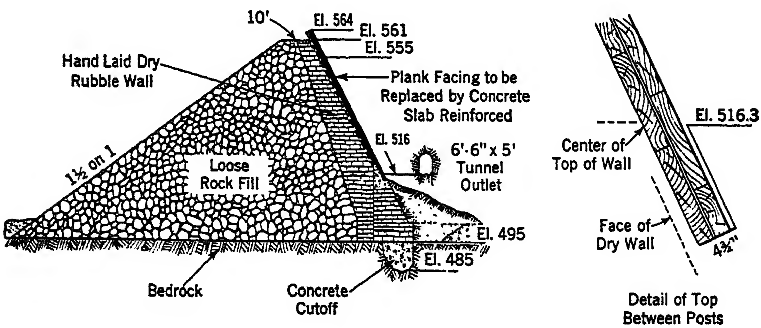

FIG. 7. Drew's Dam, Oregon. (*Eng. News, Vol. 77, p. 100.*)

facings are often economical in remote locations and are particularly applicable where the variation in the elevation of the water surface is not great. These facings are usually from one to three layers of 2 by 12 in. or 3 by 12 in. plank laid parallel to the dam axis and spiked to sills, say 8 by 8 in., embedded in and anchored to the rubble cushion.

Steel Facing. Steel plates have been successfully used to provide the impervious membrane on the upstream face of rock-fill dams, as in the Penrose-Rosemont Dam, Colorado (31), and the Skaguay Dam of Southern Colorado Power Co. (28). Fig. 8 shows the means adopted at the Penrose-Rosemont Dam for anchoring and joining the steel facing plates. Expansion joints of the U type

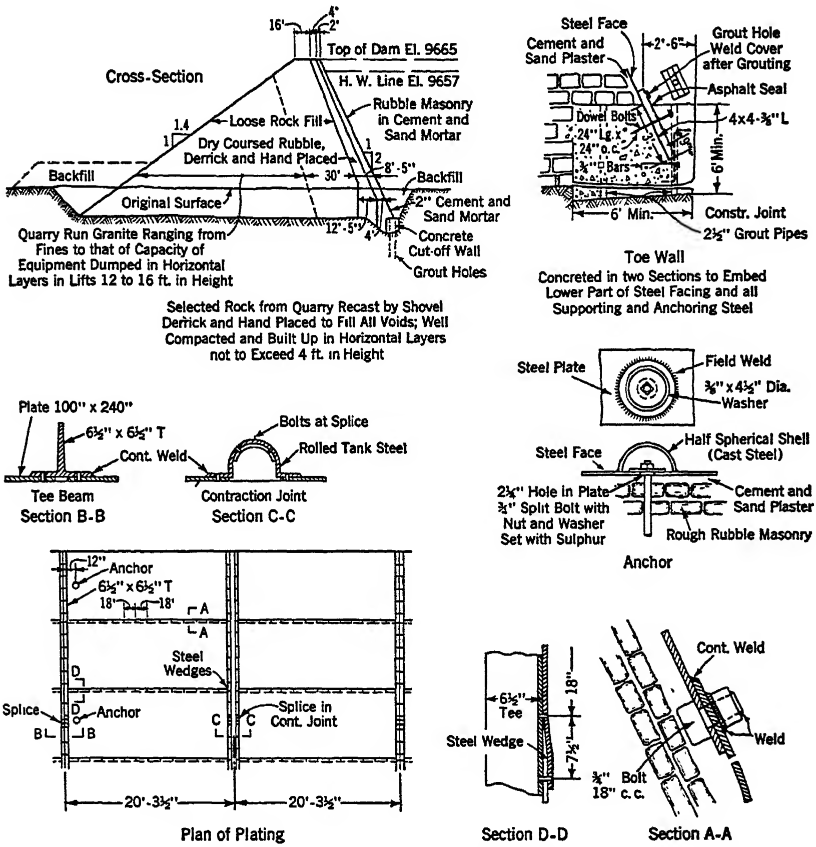

FIG. 8. Penrose-Rosemont rock-fill dam. Note the steel plate upstream face. (*Eng. News-Record, Vol. 108, p. 761.*)

are generally used and some means must be adopted for anchoring the plates to the rubble backing. The plate joints may be riveted and caulked or welded. Facing plates range from ¼ to ½ in. in thickness and should be of copper bearing steel to reduce corrosion. Protection from the action of water and air is secured by painting all surfaces with bituminous paints. After 30 years of use, the Skaguay Dam, which was painted on the face only, shows only very slight corrosion.

Concrete Facing. The general practice is to use reinforced concrete to provide the impervious membrane on the upstream face of rock-fill dams. This membrane may be a single sheet of concrete or it may be made laminated with two sheets. Most recent practice of the single sheet has secured good results and may now be considered standard practice in the United States.

It has been found that a concrete facing can be made safe against water pressure up to 300 ft with a thickness equal to about 1 per cent of the height with a minimum thickness of 12 in. (1). The reinforcement is usually made the same in both directions and equal to about 0.5 per cent of the cross-section of the slab. It is usually placed in the center of the slab except where thickness exceeds about 2 ft, in which case two layers may be used, one near each face.

The concrete may be placed in 30 to 60 ft squares with expansion joints to permit adjustment under settlement of the rock fill, or it may be poured as one large mat with construction joints only between pours and with reinforcing continuous across the joints. The relative length of the upstream facing to its vertical height plays an important part in determining the method of pouring the facing slab. In high dams in narrow canyons, the facing should be divided into squares by expansion joints to accommodate the larger settlement movements to be expected.

At Bucks Dam, 122 ft high, a reinforced concrete facing over 1000 ft long was poured directly on the rubble backing without expansion joints and after 8 yr there were no cracks other than a few hair cracks.

With regard to cracking of the concrete facing slab at Salt Springs Dam (Figs. 4, 9, and 10), I. C. Steele, Chief of Division of Civil Engineering, Pacific Gas & Electric Co., writes as follows: "Since its completion in 1931 the concrete facing of Salt Springs Dam has been frequently inspected from the crest (elevation 3958.5) to minimum reservoir level at elevation 3710. The water surface never has been sufficiently lowered to permit of face inspection along the cutoff wall in the river channel. A large number of narrow closely spaced cracks and a few large cracks occurred in the facing adjoining the canyon walls. Generally, these are several feet from the cutoff wall and are largely confined to an area 20 ft wide, roughly paralleling the periphery of the canyon and extending from a level 60 ft below the crest to the lowest point of inspection. These cracks were caused by severe differential settlements near the abutments chargeable principally to water load and to lateral settlement of the rock-fill mass from the abutments toward the center of the dam. The 1-in. space at the vertical joints proved insufficient in the central one-half of the length of the structure. High lateral compressive forces closed these openings and caused occasional spawling of the top 2 to 3 in. of the concrete slab at three vertical joints immediately below the crest. Minor repairs were made. Concrete along 120 ft of horizontal joint 162 ft below crest and adjacent to the left canyon wall was badly broken and crushed, primarily because the down-slope thrust created a heavy eccentricity of loading within the slab owing to large and differential settlements of the underlying rock fill. During 1939 the damaged sections were completely removed and replaced with new concrete. To date (1939) less than $25,000 has been expended in face repairs."

It is believed that two or more additional expansion joints, spaced about 5 ft apart, in addition to the expansion joint at the cutoff wall, should be provided along the canyon wall in the case of high dams, but the number of such joints

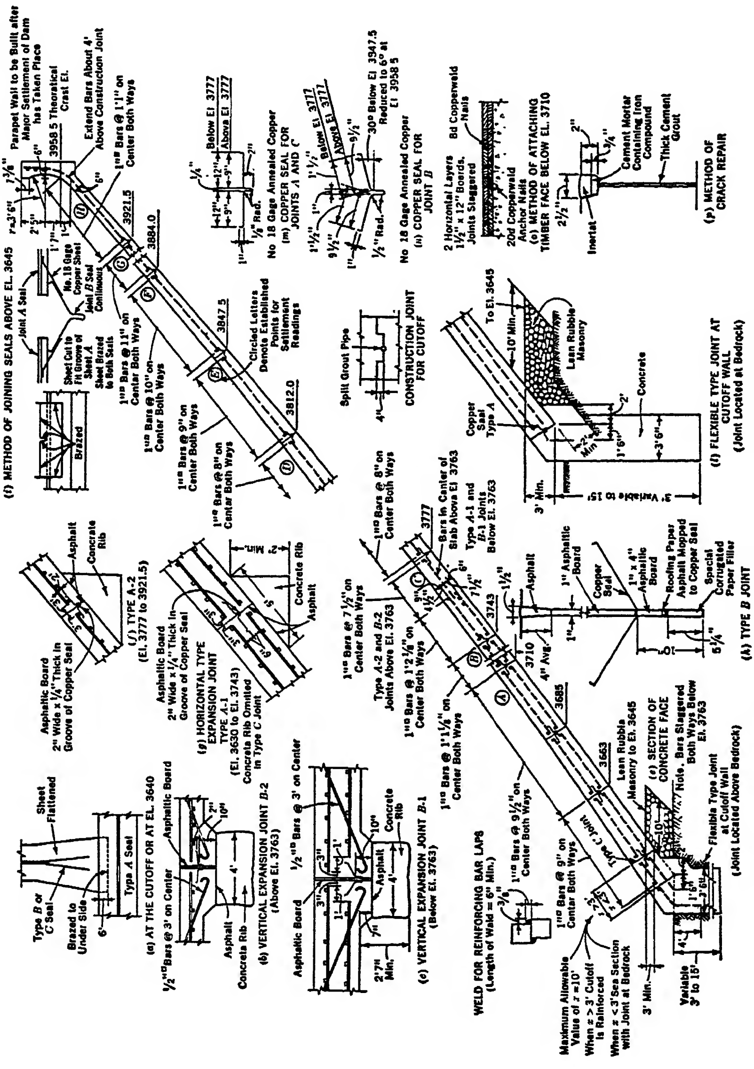

FIG. 9. Details of concrete face and cutoff wall, Salt Springs Dam, California.

may be decreased near the crest. Furthermore, vertical expansion joints close to the abutment should be built in closed position and those in the center of the dam in open position, varying gradually from the abutments to a 2 to 3 in. opening at the center to provide for the settlement movement toward the center.

The copper water stops quite generally utilized at expansion joints should be so designed that material movement may take place between adjoining blocks without rupturing the water stop. To facilitate such action a piece of asphalt board, about $\frac{1}{4}$ in. thick, or other relatively soft material, is sometimes inserted in the groove of the copper water stop and the tongue swabbed with hot tar or asphalt to prevent adherence of concrete. Great care should be exercised in placing the water stops to make sure that there are no areas of honeycombed concrete in contact with the water stop.

Earlier practice in facing construction was to bring the rubble cushion to a smooth surface by plastering the joints or by laying up a course of rubble with mortar joints, sometimes applying a bituminous coating thereon, so that the concrete diaphragm would be free to move independently of the rock settlement.

Later practice, as at Dix River Dam (5), Salt Springs (1 and 8), and other dams, has been to construct grooves in the rubble cushion approximately 2 ft deep by 3 to 4 ft wide along the lines of vertical and horizontal expansion joints. The horizontal and vertical concrete stringers on which the slabs rest should be poured in these grooves without the use of bottom forms so that they will bond thoroughly into the rock fill. They need not be reinforced. Their top must be finished to line and grade to receive the concrete slabs.

The top of the rubble cushion should be chinked with spalls sufficiently to prevent waste of concrete or loss of mortar from concrete. The concrete should then be poured directly upon the top surface of the rubble cushion. The concrete facing and a portion of the rubble cushion are thus so bonded together as to act as a monolith following the settlement of the rock fill beneath.

Some engineers feel that bonding the slab to the rubble cushion tends to prevent the free action of the expansion joints. In order that the expansion joints should act properly as such it would be necessary to use either a light wooden form or a mortar bed covered with tar paper on top of the rubble cushion and to anchor the upper end of the concrete slab to the stringer by means of a shear key and heavy reinforcing bars, leaving the lower end and sides free to move on the other stringers, the tops of which should be coated with asphalt to prevent adhesion. However, as described above, it has been demonstrated by actual construction that the concrete facing will adjust itself when poured directly on the rubble cushion. The cost of such facing is less than any other type, and successful experience justifies its use. Fig. 9 shows some of the more important details of the reinforced concrete facing used at the Salt Springs Dam in California.

8. Settlement and Sluicing. Total vertical settlement in excess of 5 per cent of the height has occurred in some rock-fill dams, and the horizontal displacement may be nearly as great. If, however, the loose rock dump fill is constructed in advance of the rubble cushion with a proper use of sluicing water, the initial settlement may be large; but subsequent settlement, after the placing of the rubble cushion and the impervious facing, should not exceed 2 per cent of the height.

In the Salt Springs Dam, height 328 ft, for instance, provision was made for a vertical settlement of 6 ft and a horizontal displacement of 4.2 ft. The actual vertical settlement at the crest has been about 2 ft. Settlement is not uniform nor is it vertical. The tendency is for movement to take place in a direction perpendicular to the water face. The proximity and configuration of the canyon walls have a marked influence on settlement at any particular point. At Salt Springs Dam the maximum total displacement took place at a distance about 40 per cent up from the base of the total height of the dam. The observed settlement for several dams is given in Table 3.

TABLE 3

OBSERVED SETTLEMENT ROCK-FILL DAMS

Dam	Height (ft)	Period of Observation (yr)	Total Horizontal Movement (ft)	Total Vertical Settlement (ft)	Total Vertical Settlement as Per Cent of Height
First Bowman	96	45	....	1.28	1.33
Dix River	270	11	1.93	2.48	0.92
Strawberry	140	19	1.60	2.11	1.50
Swift	125	22	3.10	2.93	2.34
Morena	148	..	....	1.65	1.00
Oued Kébir	115	..	....	3.10	2.70
Vannino	76	..	....	1.67	2.20
Salt Springs	328	8	1.01	1.97	0.60
Nantahala	260	2	0.40	1.00	0.40

Small chips and dust, if present to any considerable extent, will lodge between the rocks and will later sift down into the interstices of the larger rock under the action of rain falling on the dam and passing down through it. This may cause a material settlement of the dam and may cause serious movement and cracking of the impervious upstream face. Accordingly the fines should be constantly washed into the rock mass with hose streams as the fill is being made. The quantity of water which should be used for this sluicing operation will vary with local conditions. Theoretically if there are no spalls or dust, no sluicing would be required, but practically it has been found desirable to use from 2 to 4 times the volume of the dam in sluicing water. Continuous sluicing during construction will produce presettlement of the fill and thus increase stability. The elimination of lenses of small particles thus obtained will reduce local settlement when water pressure is applied on the dam. Generally speaking, the use of an adequate amount of sluicing water during construction is one of the most important factors in the proper construction of rock-fill dams. In this connection, it is of interest to note that in the original construction of San Gabriel No. 2 very little sluicing was done and as a result the first heavy rain resulted in a 12-ft settlement causing the destruction of the concrete facing (10). At Salt Springs Dam and San Gabriel No. 1 the sluicing water was twice the volume of the rock fill. At Nantahala, N. C., the ratio of water to rock fill exceeded 4.

In addition to the settlement caused by chips and dust sifting down into the interstices of the larger rocks, referred to above, settlement is also caused by the crushing of the bearing points of the rocks when the load comes on them from the weight of the rocks above and from the transmitted water pressure. In this connection it is worth noting that quarry run rock, which contains assorted sizes, provides more bearing points than big rock alone of uniform size.

The settlement is, of course, much greater in the early life of the dam and is greatest during the first few months. The dam is usually built from the canyon sides toward the center, and as the fill advances from each side it causes the rock to settle toward the center. The initial settlement hastened by sluicing should be allowed to take place to as great an extent as practicable before the construction of the impervious upstream face is started.

9. Spillways and Freeboard. It is very nearly as essential to prevent the overtopping of a rock-fill dam in time of flood as it is an earth dam. Any type of spillway suitable for an earth dam is also suitable for a rock-fill dam. In many cases the "side channel" type of spillway, where a spillway channel is cut through the ledge rock at one side of the dam, has been utilized. Sometimes the discharge from such a spillway is through a tunnel. Other things being equal a "saddle" spillway remote from the dam itself is desirable.

The spillway capacity to be provided should be determined by a careful study so that there will be no danger of overtopping the rock-fill dam. Some spillway requirements have been met by constructing a spillway over the rock fill out of timber and planking, as was done in the Beaver Park Dam in southern Colorado. The Laguna weir in the Colorado River, built by the United States Reclamation Service, is essentially a rock-fill overflow dam; but its height is 24 ft and it has a bottom width of more than 200 ft and a downstream slope of 1 on 12. Many of the early spillway dams in the eastern part of the United States were rock-filled timber cribs with planked upstream and downstream faces. Some of these are more than 100 yr old and are still in service. However, this is no precedent for using a modern rock-fill dam, which is devoid of restraining timber cribs, as a spillway. Paving or planking the rock fill to form a spillway over the top of a high rock fill is inadvisable because a rupture of the spillway paving may cause erosion or even the destruction of the rock fill.

Most failures of rock-fill dams have occurred by overtopping. In addition to ample spillway, ample freeboard must be provided, as any overtopping where the water attains an erosive velocity is apt to result in failure. In determining freeboard above pond elevation during the maximum extimated flood, it is not necessary to use quite as high a factor of safety as in a comparable earth dam because if spray passes over the top of the dam it will not injure the downstream face.

Salt Springs Dam. The Salt Springs Dam is a good example of modern practice in the design and construction of a high rock-fill dam with an impervious upstream face formed by a diaphragm of reinforced concrete slabs. This dam is one of the largest rock-fill dams (3,170,000 cu yd) as well as one of the highest (328 ft). Fig. 10 shows a typical cross-section and Fig. 9 shows some of the essential

details. It will be noted that the rock fill has natural rock dump slopes and that the thickness of the rubble cushion of hand- and derrick-placed rock is relatively moderate, being about 15 ft perpendicular to the slope from the bottom to the

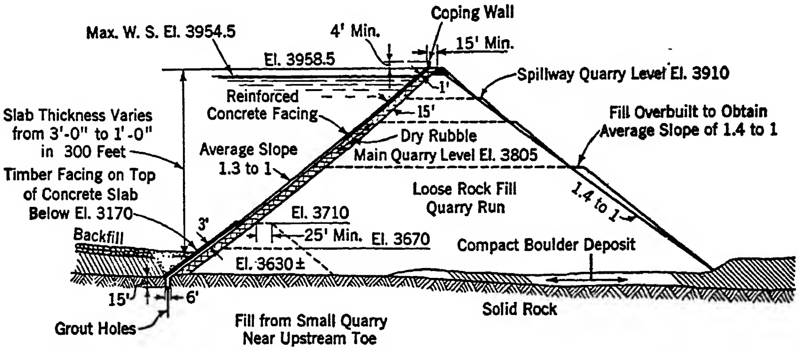

Fig. 10. Salt Springs Dam, Mokelumne River, California. Maximum cross-section. (*Courtesy Pacific Gas & Electric Co., San Francisco.*)

top of the dam. The reinforced concrete facing was laid in squares 60 by 60 ft on horizontal and inclined concrete stringers located at the expansion joints and embedded in the rubble cushion.

10. Core Wall Type of Rock-Fill Dam. Rock-fill dams are sometimes built with a core wall of concrete or reinforced concrete. A steel diaphragm, with a thin protecting layer of concrete on each side, has also been used as a core wall in such a dam. The core wall is usually placed in about the center of the dam and the loose rock fill dumped on each side of it. The upstream half of the rock dam is thus submerged, which decreases the stability of the dam, requiring additional material downstream to take the full horizontal thrust of the water pressure. The center diaphragm cannot be repaired in case of settlement or cracking. The Lower Otay Dam, in California (24), was of this type (see Fig. 11). This dam was built in 1897 to a height of 135 ft and, after nearly 20 yr of service, failed by overtopping in 1916. The side slopes of 1 on $1\frac{1}{4}$ are steeper than would be considered conservative today for a dam of this type.

A dam of this type is practically an earth dam of excessively pervious material, depending absolutely on the imperviousness of the core wall. In turn, the stability and safety of the core wall depends on the supporting power of the loose rock fill. It is essential that there should not be much earth or debris in the rock fills, as this leads to undue settlement, which may unbalance the pressures on the core wall to such an extent as to cause serious cracks and distortion, and, in an extreme case, bring about failure.

11. Composite Type of Rock-Fill Dam. This type consists of a rock fill on the downstream side of the dam and an earth fill on the upstream side. Such a dam, when properly constructed, produces a very stable and satisfactory structure. The earth fill furnishes the watertight portion of the dam, an intermediate section provides a filter, and the rock fill forming the downstream portion pro-

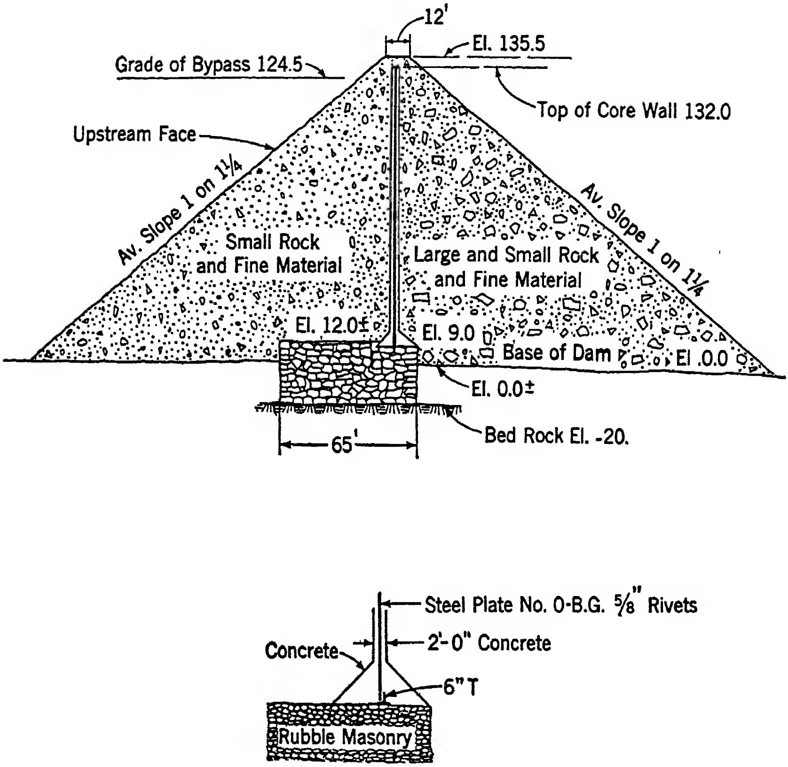

FIG. 11. Typical cross-section of Lower Otay Dam. (*Courtesy George Cromwell, Chief Engineer, San Diego County Water Co., Los Angeles, Calif.*)

NOTE: Failed by overtopping in 1916.

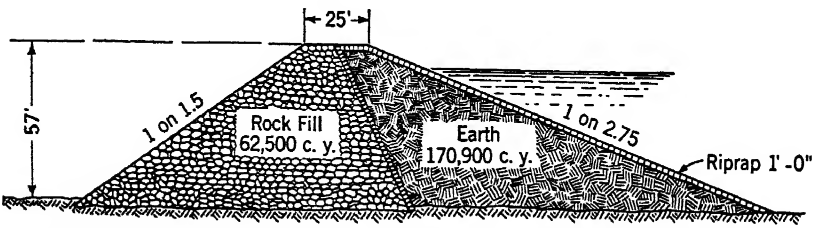

FIG. 12. McMillan Dam, Pecos River, New Mexico. (*Information by courtesy of Bureau of Reclamation, Denver, Colo.*)

vides ready drainage for seepage water and adds a greater degree of stability to the structure than would generally be provided by an equivalent amount of earth.

McMillan Dam (Fig. 12), known also as Pecos Valley Dam No. 2, the Inland Dam (Fig. 13), and the Glenville Dam (Fig. 14) are examples of the composite type rock-fill dam. In constructing such dams, the rock fill should be carried up ahead of the earth. Where the earth-fill and the rock-fill portions of the dam join, the voids between the big stones should be chinked up with smaller stones and spalls and then several heavy layers of graded crushed stone or gravel should be placed on the upstream surface of the rock fill before the earth fill is placed against it. The proper use of these layers of crushed stone or gravel is a matter of the greatest importance to the permanent safety of such a dam, as there must be no penetration of finer material into coarser material due to water pressure or seepage.

A suitable gradation for such a composite dam of moderate height would be the following:

1. On the chinked-in surface of the dumped rock place a layer 3 ft thick of screened gravel or crushed stone having a minimum size of ½ in.

2. On the above layer place another 2 ft thick, grading between ¼ in. and ½ in.

3. On the above layer place one 18 in. thick of coarse sand which with all pass a ¼-in. sieve. On top of this third layer place the earth fill, placing the most pervious portion of the material near the downstream limits and most impervious near the upstream face. If the dam is over 100 ft high greater thicknesses of filter may be desirable in its lower portion.

In effect this measure produces a filter and prevents the earth from being carried away through the large interstices of the rock fill by the action of seepage water and rain. A suitable substitute for the graded layer filter is a single, much thicker layer of run of bank gravel which contains suitable proportions of the various sizes. Such a layer should seldom be less than 10 ft thick because some of the sand will penetrate into the rock fill. Such movement, however, is quickly blocked by the larger sizes of gravel.

In the construction of the Inland Dam, near Birmingham, Ala., a composite type (Fig. 13), a considerable portion of the sandstone, which formed the rock-fill part of the structure, was crushed and graded in order to produce the required filter effect and eliminate fines.

12. Earth Core Type of Rock-Fill Dams. A conventional type of rock-fill dam having an impervious element consisting of an earth core in the center of the dam is indicated in Fig. 16. A somewhat unique type in which the core is considerably inclined upstream, as indicated in Fig. 15, was constructed on the Nantahala River in 1941, in western North Carolina, by the Nantahala Power & Light Co. The Nantahala Dam was designed and constructed under the direction of J. P. Growdon, Chief Hydraulic Engineer of the Aluminum Co. of America. The arrangement of the inclined impervious diaphragm of earth, with filters both above and below it, is believed to be original with Mr. Growdon.

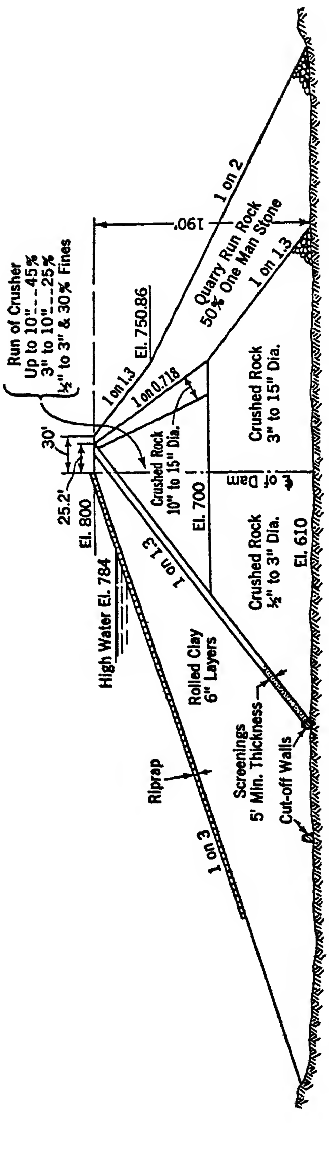

FIG. 13. Inland Dam, City of Birmingham, Ala.

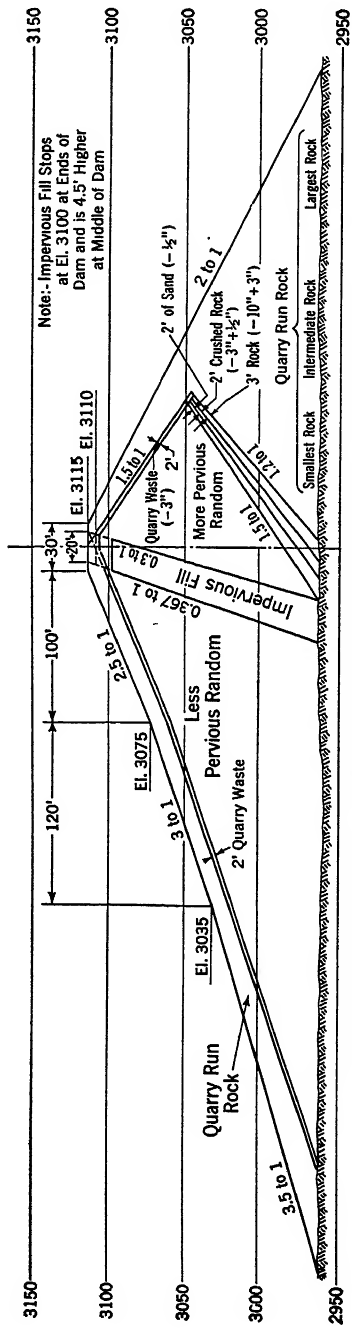

FIG. 14. Glenville Dam. (*Courtesy Nantahala Power & Light Co., North Carolina.*)

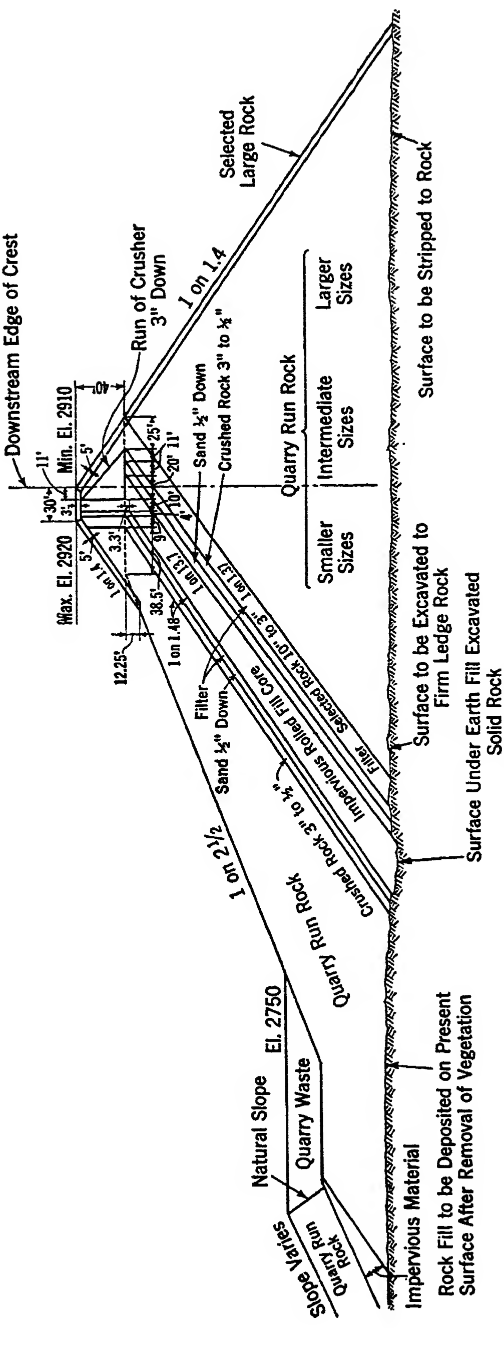

FIG. 15. Nantahala Dam. (*Courtesy Nantahala Power & Light Co., North Carolina.*)

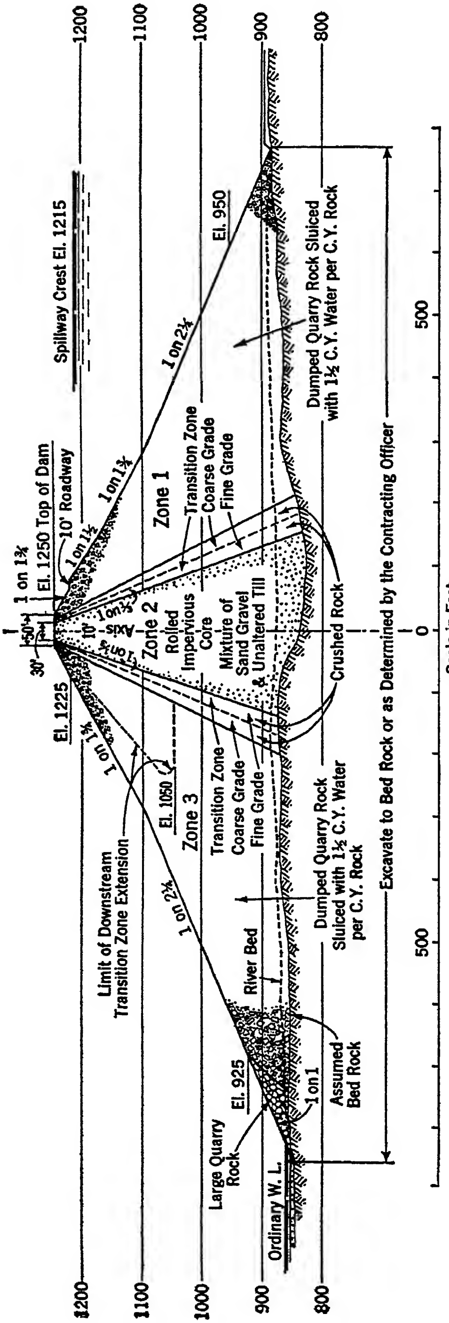

FIG. 16. Stevens Dam. (*Courtesy U. S. Engineer Office, Seattle, Wash.*)

This type of dam is particularly adapted to locations where there is a plentiful supply of good rock fill and, at the same time, sufficient earth for the core. In addition, the earth core possesses certain advantages over the concrete slab or other conventional facings frequently used for rock-fill dams. It can yield readily to settlement of the rock fill without damage; it is self-healing with respect to any cracks which might be formed. Because of this feature dams of this type often have less seepage than rock-fill dams with a concrete face. The seepage at Nantahala, for instance, is only about 6 cu ft per min.

As shown in Fig. 15, the core is protected on both sides by filter layers. The downstream filter prevents loss of core material by piping due to pressure of reservoir water, and the upstream filter affords similar protection against reverse flow when the reservoir is drawn down.

The required thickness of core depends on the permeability of the material. When highly impervious fill is available, a very thin blanket would, theoretically, suffice. However, there are practical considerations which also govern, such as adequate space for spreading and rolling and the provision of an ample mass to allow for deformation due to settlement of the rock fill. These latter considerations determined the thickness of the Nantahala core.

Similarly, the thicknesses of the filter layers will, in general, be governed by practical construction requirements. Some "seasoning"—migration of fine particles near the layer boundaries—is certain to take place, and an ample margin must be allowed for this, as well as for the inaccuracies in placing, which are unavoidable in rapid construction. Tests on efficiency of available filter material are usually made. (See Art. 24, Chapter 17.)

Considering the general case, it is obvious that the core could be placed anywhere within the section from a position at the center, as in Fig. 16, to a position nearly parallel to the upstream slope, as in Fig. 15. In the vertical position, the water pressure is transmitted horizontally to the downstream rock fill, so that generally, in order to secure adequate stability, the downstream slope must be flatter than the angle of repose. Moving the core from this position to progressively flatter positions upstream permits a steepening of the downstream slope but requires a corresponding flattening of the upstream slope to provide protection against upstream sloughing during drawdown.

In an investigation of the optimum position of the core, Glennon Gilboy has found that shifting the core over the full possible range will not change materially the total yardage of the dam. He concludes, therefore, that economy of design appears to rest on considerations of efficient construction rather than on total yardage.

He points out that locating the core in such a position that the upstream face of the downstream rock fill can be built on its angle of repose will offer the following economies.

a. Sufficient stability against direct water pressure will be obtained with the downstream face at its angle of repose, thus avoiding, for the entire downstream rock fill, any expensive rehandling when placing.

b. The downstream fill can be placed well in advance of the core and take a large part of its settlement before the core is placed upon it.

c. The downstream filter can be placed at its angle of repose, avoiding the use of batter boards or saw-tooth construction in this very important element of the dam.

The crucial feature of design of a dam of this type is the stability of the upstream slope when subject to rapid drawdown of the reservoir. Proper methods for testing for stability under such conditions are described in Chapter 18.

13. North African Rock-Fill Dams. French engineers have designed and constructed a number of rock-fill dams in Algeria (6); and modern practice there in their design and construction is so different from that in America that it is desirable to take note of it here. The partial failure of the Oued Kébir Dam in 1929, composed largely of a dumped rock fill, due to the shearing of its centrally located hollow multiple-arch reinforced concrete core wall (19) led to a general revision of the criteria of design and construction.

Recent rock-fill dams in Algeria have been constructed entirely of derrick- and hand-placed stone. Essentially they are dry rubble dams with special care taken in the use of selected rock near the upstream face. Voids are thus reduced to 26 to 32 per cent of the total mass as contrasted with 35 to 45 per cent in American rock-fill dams.

Upstream slopes are usually somewhat steeper than 1 on 1 and downstream slopes may be as steep as 1 on 1¼. Selected carefully placed stone is used at both faces.

The impervious diaphragm is usually located at the upstream face, and refinements in design, unknown in America, are used to insure watertightness and accessibility for inspection and repair. It appears to be usual practice to locate an inspection gallery in the cutoff wall at the point where it connects to the impervious upstream face.

The unusual precautions taken to insure a tight impervious diaphram on the upstream face are illustrated by the measures adopted at the Ghrib Dam (height 233 ft). Here a course of rubble laid in mortar was used at the upstream face. On top of this was placed a 3-in. layer of special porous concrete. The top surface of this concrete was then sprayed with bitumen and a 2⅜ in. layer of bituminous concrete placed and rolled. On top of this bituminous concrete a second layer of the same material and the same thickness was placed. On top of this bituminous layer was placed a thin layer of mortar, and then finally a layer of reinforced concrete about 4 in. thick divided into slabs about 10 by 6½ ft, but with the mesh reinforcement continuous. The top layer of reinforced concrete, which is intended to serve as heat insulation for the bituminous concrete underneath, is sprayed with water during very hot weather.

In cases where seepage downstream from the cutoff walls is anticipated, extensive precautions are sometimes taken to drain away the seepage water at velocities which will not erode the foundation. Thus at Bou Hanifia, where a marly sandstone overlies an impervious marl, the foundation being badly faulted, an extensive inverted filter and drainage system is utilized, following a design by Terzaghi.

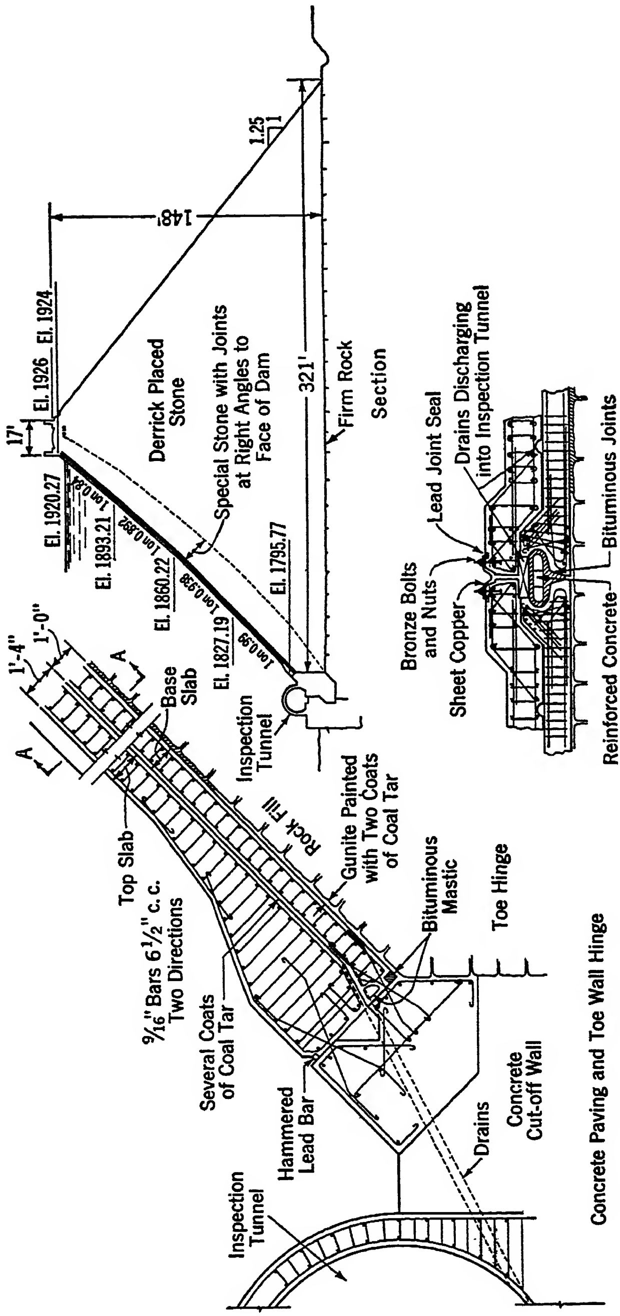

FIG. 17. Bakhadda Dam, Algiers, North Africa.

The Bakhadda Dam (Fig. 17) in Algeria was built entirely of derrick-placed stone and was designed to have two concrete facings. The lower facing was built and the dam was placed in operation in order to allow the settlement under water load to take place. The leakage was found to be practically nonexistent with only the lower facing installed. However, on account of possible earthquake disturbance, the original plan was carried through and the upper facing was poured after the two facings were thoroughly separated by bituminous paint. A system of drainage by semicircular conduits was installed and connected with the observation gallery. Fig. 17 shows a cross-section of this dam and the elaborate details of hinge and expansion joints.

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CHAPTER 21

STEEL DAMS

1. General. The use of structural steel for the building of dams has not been very extensive in this country. Only three (1, 2, 3, 4) [1] important structures have been of this type. The Ash Fork Dam in Arizona, Fig. 4, built in 1898, and the Redridge Dam in Michigan, Fig. 5, built in 1901, have given satisfactory results. The Hauser Lake Dam in Montana, Fig. 6, built in 1906, failed after 1 year of service. Its failure was attributed to undermining of the foundation by leakage through or under the steel sheet pile cutoff and was in no way a reflection on the superstructure. However, an opinion has been expressed that the leakage may have been caused by the pulling of the piling, which was used as an anchorage as explained in Art. 7.

Although the aversion to steel dams has been mitigated to a great extent by the excellent records of the Ash Fork and Redridge Dams, there still remain the following defects, some of which may be immaterial but which nevertheless have influenced in the past their rejection as possibilities.

1. Steel is not considered as permanent as concrete, particularly in the case of solid concrete dams.
2. Steel requires greater and more constant maintenance than concrete.
3. Steel dams, being lighter, are not as adaptable to absorb the shock from vibrations of spilling water.
4. The types of steel dams which have been built require anchoring to the foundation, a procedure which is possible but which is not considered good practice for concrete dams.
5. Considerable concentration of bearing stresses, as in Fig. 3.

The advantages of steel dams are:

1. Greater speed in construction.
2. Claimed less cost.
3. Stresses more determinate.
4. Greater flexibility to resist unequal settlement without excessive leakage.
5. Not affected by frost action.
6. Modern welding processes permit leaky joints to be more easily repaired than in hollow concrete dams.

2. Design. The theory for the design of steel dams is the same as has been described for hollow concrete dams in Chapter 14. There are two general types of gravity steel dams.

[1] Numbers in parentheses refer to Bibliography, Art. 11.

The direct strutted type, as shown diagrammatically in Fig. 1, is the simplest. It carries the load directly from the deck to the foundation through inclined struts.

The cantilever type consists of variations of the direct strutted type, in which the section of the bent supporting the upper part of the deck is formed into a

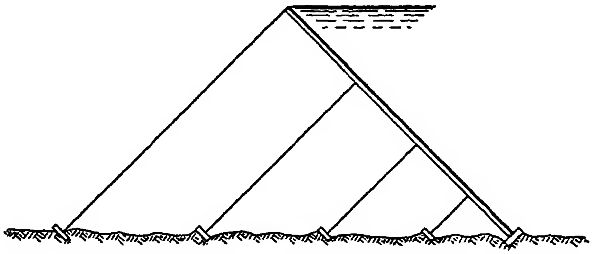

FIG. 1. Direct-strutted type.

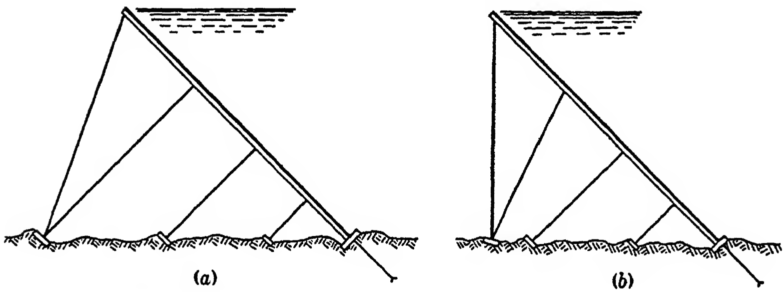

FIG. 2. Cantilever type.

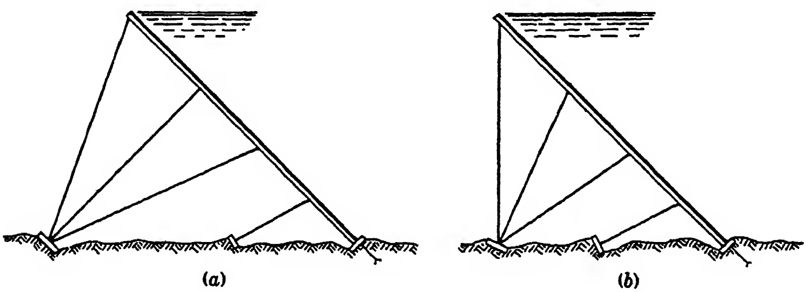

FIG. 3. Cantilever type, struts flattened.

cantilever truss, as shown in Figs. 2 and 3. This type introduces a tensile force in the deck girders which must be taken care of. This may be done in three ways.

1. The deck girder may be anchored into the foundation at the upstream toe.

2. Hovey (5, p. 26) suggests that the tension at the upstream toe, and hence the necessary anchorage, may be reduced by flattening the slopes of the lower

struts in the bent as shown in Fig. 3. That is, as the water load normal to the deck girder is transferred to the flattened strut, a component stress is induced parallel to the deck girder which will reduce the tension. This adds considerably to the expense, however, because the struts must be not only longer but also of a heavier section.

3. The entire bent may be framed together rigidly so that the moment of the weight of the water on the lower part of the deck may be utilized to offset the moment of the cantilever. Bainbridge says (6), however, that the cost of this bracing would be excessive.

Bainbridge also states that there is little difference in cost between the direct strutted type of Fig. 1 and the cantilever type of Fig. 2. However, all three of the steel dams which have been constructed in this country and several which have been designed are of the cantilever type requiring anchorage at the upstream toe.

The cantilever type requires the least width of base. This was one of the considerations affecting the choice of that type for the Redridge Dam. The cantilever type having a vertical downstream face is more adaptable to spillway dams. However, it is not known whether or not this feature or the question of economy dictated the choice of the cantilever type for the Ash Fork Dam. The Hauser Lake Dam was of the cantilever type but also had an extensive apron to carry the overflow. Of course, where satisfactory anchorages are not obtainable, the direct strutted type must be used.

No attempt will be made in this book to design the steel members of the dam. For this, a standard textbook on structural steel design should be consulted. For an example of a design of a 100-ft steel dam, reference may be made to a paper presented before the Western Society of Engineers by F. H. Bainbridge (6).

3. Slope of Face. As in the case of hollow concrete dams, the upstream face of a steel dam is built on a slope in order to take advantage of the weight of the water for stability. Hovey has shown (7) that a rigid complete right triangular frame similar to the shape illustrated in Figs. 2*b* and 3*b* would be in exact equilibrium for rotation about the downstream toe if the face were inclined at an angle of 54° 44′ with the horizontal, neglecting the weight of the structure itself. The Ash Fork and Redridge Dams, however, were both built with slopes of 45°. The Hauser Lake Dam was constructed with a 1 on 1½ slope. Bainbridge says (6) for economy the slope should never be flatter than 45°, since, as the angle decreases below 45° the weight of the face plates and deck girders is increasing faster than the cosecant of the angle, and the weight of the supporting struts is also increasing. Hollow concrete dams are usually built with approximately a 45° slope.

4. Face Plates. The deck is made up of cylindrically curved plates, as illustrated in Fig. 4, placed concave to the water. These act practically as a suspension system (7) and will be found more efficient and economical than the use of flat or buckle plates. These plates may be either riveted and caulked or welded to make a perfectly watertight face.

To provide for a reasonable amount of corrosion of the surface without too large a percentage reduction in thickness, a minimum thickness of plate of $3/8$ in.

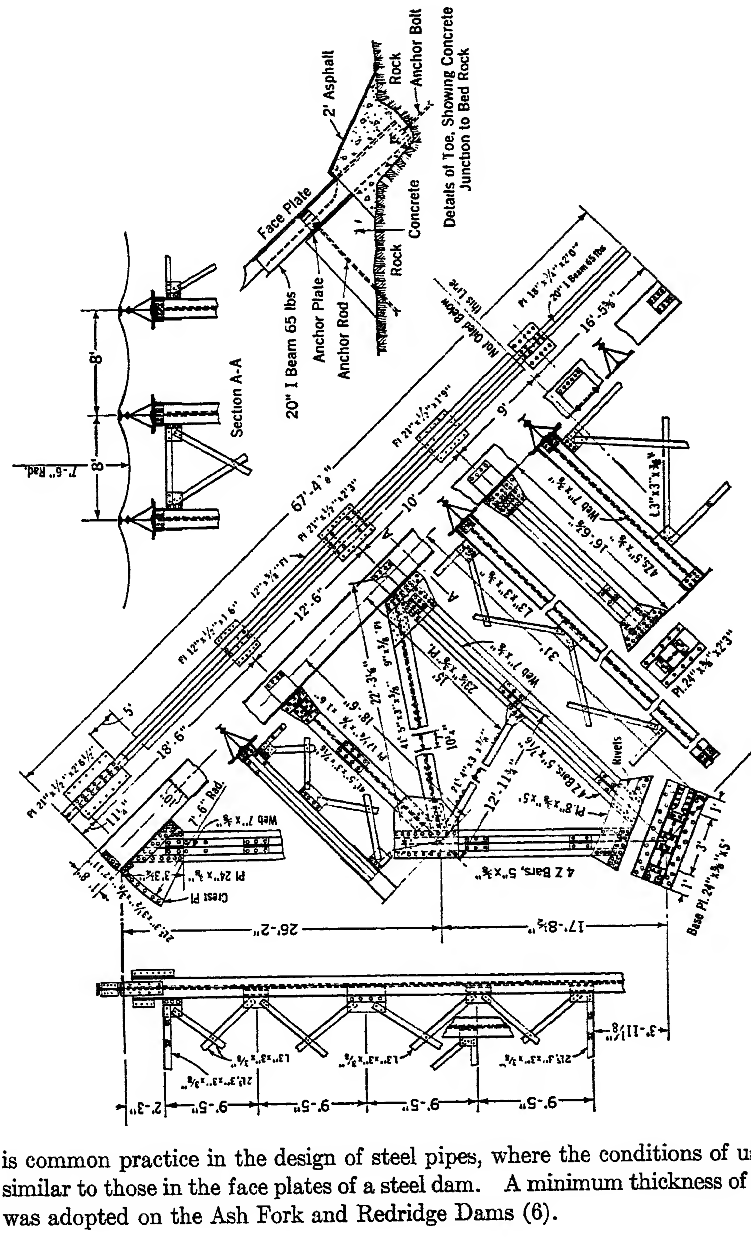

Fig. 4. Details of steel dam at Ash Fork, Ariz. (*From J. Western Soc. Engrs., Vol. X, 1905.*)

is common practice in the design of steel pipes, where the conditions of use are similar to those in the face plates of a steel dam. A minimum thickness of $3/8$ in. was adopted on the Ash Fork and Redridge Dams (6).

The tension at the connection of the plates to the deck girders will be balanced throughout the dam except at the abutments. Here anchorage must be provided to prevent bending in the web of the last deck girder.

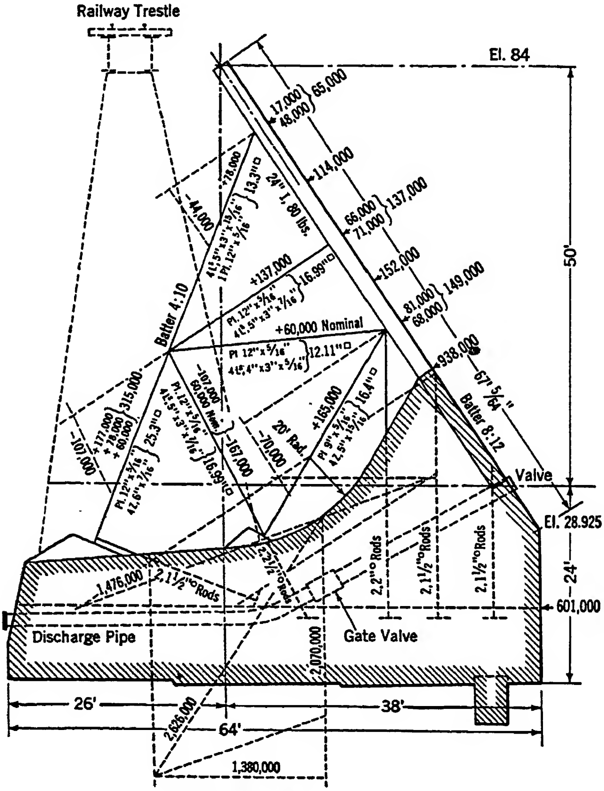

FIG. 5. Cross-section of Redridge Steel Dam in Michigan. (*From J. Western Soc. Engrs., Vol. X, 1905.*)

5. Deck Girders and Bents. The deck girders supporting the face plates are most economically designed as continuous beams (7) with struts spaced at increasing intervals from toe to crest. These struts, together with the girders, are arranged in a series of bents. Bracing is provided between bents, which are grouped together as shown in Fig. 7.

The spacing of the bents is a matter of economy. At Ash Fork and Redridge Dams the bents were placed 8 ft center to center. At Hauser Lake Dam a 10-ft spacing was used.

6. Expansion and Contraction. The bays between bent groupings are left free of bracing to allow the curved plates to take up any lateral expansion or contraction. It is essential to provide against any such movements at the abut-

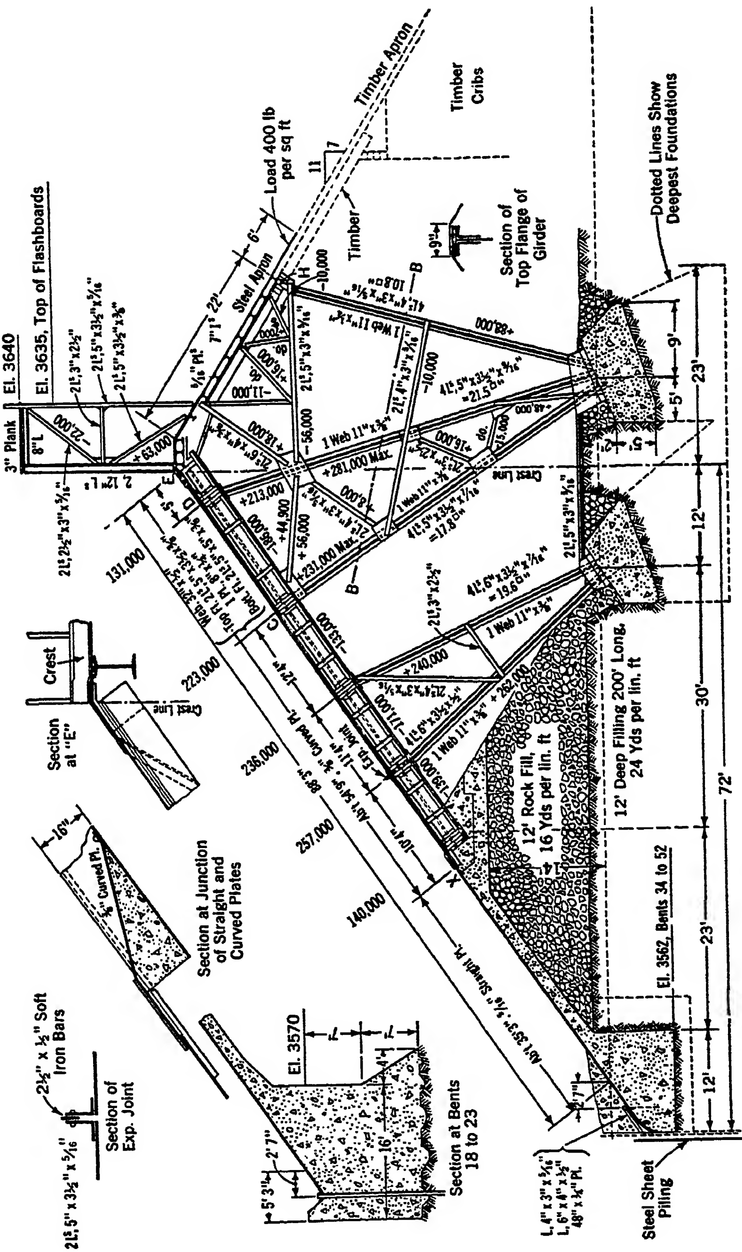

FIG. 6. Details of Hauser Lake Steel Dam. (*From Eng. News, Vol. 58, p. 508.*)

ments and foundation with good anchorage in order that these connections will not be loosened and cause leakage.

7. Foundation. Good anchorage for taking up the tension in the deck girders of the cantilever type steel dam and for restraining temperature movements at the abutments and the foundation can be obtained by drilling into the rock and grouting anchor bars in place. Plain rods can be grouted in holes to develop their full strength. Tests in the field can be conducted to determine the proper details.

The anchorages must engage a sufficient weight of the foundation to balance the existing tension with ample factor of safety. The estimated hydrostatic uplift at the end of the anchors must be deducted to determine the effective weight

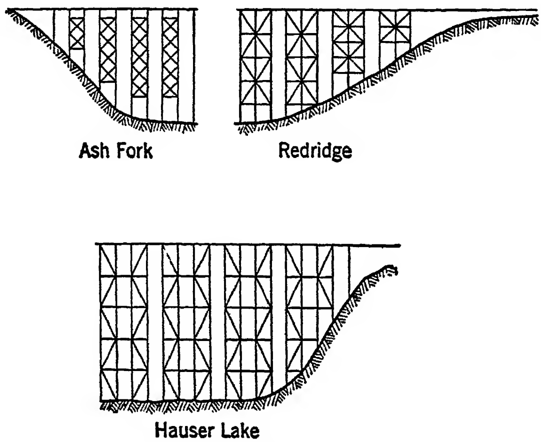

Fig. 7. Types of bracing.

of the foundation for that purpose. Direct tension in the foundation should be neglected but each anchor can be assumed to engage a wedgeshaped portion of the foundation with central angle depending upon the nature of the rock. This is mainly a matter of judgment. Consolidation grouting, as explained in Art. 7 of Chapter 3 at the top of the grouted cutoff, will materially improve anchoring in fissured foundations.

In the Redridge Dam, the steel structure is anchored to a concrete base, thus stabilizing the entire structure. Steel sheet piling driven for a cutoff may provide sufficient anchorage. This method was used for a section of the Hauser Lake Dam (3).

It is essential to make all connections to the abutments and foundations watertight. If there is to be a concrete cutoff wall, the face plates should be buried in the concrete. On solid rock it is best to build a low concrete wall, well anchored to the rock, and embed the face plates in the concrete. The plates may also be welded to a steel sheet pile cutoff. In order to make the connections at the foundation, the curved face plates are replaced by flat steel plates. These are connected to the curved plates by means of a segment cut to fit and welded in place.

The flat plates may then be bent down into the concrete wall or against the steel sheet piling.

The remainder of the foundation problem is no different from that described for concrete dams. However, in horizontally stratified rock, the horizontal shearing resistance should be assumed to be limited to the width of the base of the individual struts unless such bases are deeply embedded.

There is, of course, no uplift on the bases of the struts. However, the problem of eliminating or balancing uplift on horizontally stratified planes below the base is no different from that for a solid concrete dam. (See Art. 5*h* of Chapter 7 and Art. 17 of Chapter 3.)

8. Durability and Painting. The Ash Fork and Redridge Dams were built in 1898 and 1901, respectively. Up until 1935 the Ash Fork Dam had been repainted on an average of each 7 to 9 yr and the Redridge Dam had been repainted only once (5). Both dams are claimed to be in excellent condition. It is evident that there will be little question of the durability of the steel provided it is adequately protected by paint and that paint is properly maintained. It is entirely possible that, in the future, with the development of economical noncorrosive steels, the need for vigilant maintenance of a protective coating of paint will be eliminated.

9. Quantities and Costs. Bainbridge has computed that the quantities in steel dams average about 12,500 lb per linear ft of dam for a dam 100 ft high (6). He states that the Ash Fork Dam, 42 ft high, averaged about 2000 lb per linear ft of dam for face plates and deck girders and about 1500 lb per linear ft of dam for struts and bracing.

Published estimates of steel dams indicate a lower cost than that of any type of concrete gravity dam for the same site (6, 7, 8).

Estimates by Stanley (8) indicate that the higher the dam the greater the advantage of steel with respect to cost. This fact has also been noted (6, 7) by Bainbridge and Hovey.

10. Steel Arch Dams. Arch dams composed of steel are feasible, but to the author's knowledge they have never been built. A dam of this type has been proposed by Stanley (8).

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CHAPTER 22

TIMBER DAMS

1. Advantages of Timber Dams. Timber dams were formerly much used in this country, and are still built in sections where transportation is difficult and timber plentiful. Under such circumstances they are entitled to serious consideration as a competitor of the concrete dam.

The life of a well-built timber dam has been variously estimated at 20 to 30 yr. However, in this case, as in many similar ones, it is difficult to estimate the life of a structure that is properly maintained. Dams reputed to be 80 to 100 yr of age have been cited; but, in such instances, probably a very small percentage of the original timber remained.

The maintenance charges for timber dams are large, particularly at sites where large floods and ice runs are frequent. Leakage is frequently very great, and the leaks are often exceedingly difficult to repair if the dam is relatively high and if a drawdown of the pond for repairs seriously affects operation. The large maintenance charges and leakage have created a prejudice against this type of dam. Hence, even in sections where virgin timber is plentiful, the use of timber dams today is rather the exception.

However, timber dams are frequently built at considerably less first cost than concrete dams and are often adopted for this reason when money is scarce. As the maintenance and depreciation charges on concrete dams are practically negligible, the interest charges on a concrete dam, theoretically, must be less than the sum of the interest, maintenance, and depreciation charges on a timber dam to warrant the adoption of the former type. However, there are doubtless many instances in which a concrete dam has been constructed, although true economy would have dictated the selection of a timber structure.

2. The A-Frame Type. In Fig. 1 is shown a type of timber dam known as the A-frame type. It is generally built of squared timbers and planks and is not rock-filled. For its stability it depends on the weight of water on its deck and the anchorage of the sills to the foundation. It is probably the ancestor of the reinforced, flat-deck, hollow type of concrete dam. The deck makes an angle of 30° or less with the horizontal.

The sills, *a*, are first fastened to the ledge rock by wedge bolts or anchor bolts, preferably grouted in. The struts, *b*, are then framed to the sills and held in place by cross-bracing and batten blocks. The wales, *c*, are then placed, the entire structure being thoroughly drift-pinned together. These bents are placed from 6 to 12 ft apart, according to the height of the dam and the size of timbers used. Across the bents are placed the studs, *d*, to which the lagging, *e*, is nailed.

The lagging should be either tongued and grooved or lapped and should not be less than 2-in. stuff.

3. The Rock-Filled Crib Type. In this type of timber dam, cribs of round or squared timbers are drift-bolted together, filled with rock fragments or boulders, and topped by a plank deck. The timbers are usually spaced about 8 ft centers

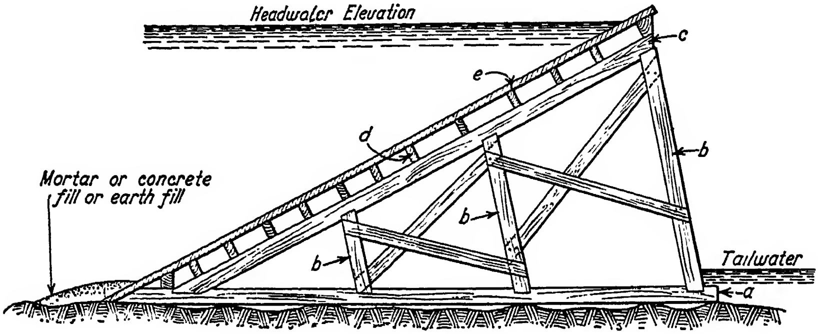

FIG. 1. A-frame type of timber dam.

both ways. The bottom timbers of the cribs are often pinned to the rock foundation if the site is not submerged. Fig. 2 shows a typical dam of this kind; but many different forms have been adopted.

For rock foundations, the shape of section indicated in Fig. 2 is frequently altered to resemble that of the A-frame type, in order to take advantage of the

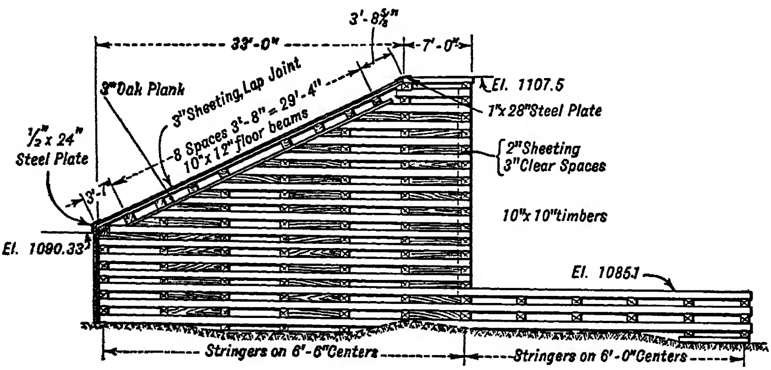

FIG. 2. Rock-filled timber-crib dam at Ocoee, Tenn.

weight of the water on the sloping deck. This obviates the necessity of the rock fill for stability against overturning, but the close crib work provides for a more substantial body of the dam than that indicated in Fig. 1. As in the A-frame type, it will then be necessary to anchor the base to the rock to prevent sliding. For low dams on soft foundations, where erosion from overflow would be serious, this section is usually reversed, having a nearly vertical upstream face and a long,

sloping downstream face, frequently stepped, in order to drop the water without great disturbance. Between these extremes many shapes of section have been adopted, some having both upstream and downstream faces sloping or stepped as in Fig. 4.

4. The Beaver Type. Another type of timber dam, which is used infrequently and only for low heads, is the beaver-type dam. Round timbers are used for the bents as in Fig. 3. The upstream slope of such a dam should not be steeper than

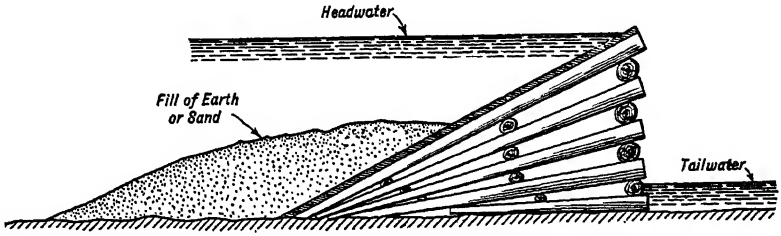

FIG. 3. Beaver type of timber dam.

1 on 2. The butts of the timbers all point downstream. Between the butts are placed spacer logs, which are drift-pinned to the other logs. Also, the tips of the timbers pointing upstream are drift-pinned together and the bottom timbers are fastened to the foundation with anchor bolts, if possible. There is usually a plank deck. Sometimes a mat of brush or the branches of the trees are used to take the place of the plank deck.

5. Stability of Timber Dams. The theory of design of masonry dams, given in Chapters 7 and 8, is also applicable to all types of timber dams. While uplift is not present in any type of timber dam, the submerged weight should be used below the elevation of tailwater. The effective weight of submerged rock fill is

$$w_s = (w - 62.5)(1 - p) \qquad [1]$$

where w is the weight of 1 cu ft of solid stone in air and p is the percentage of voids in the fill. It must be remembered that the percentage of voids in the stone fill, in the small spaces between the layers of timber in the crib work, may be very large, depending upon the care with which the fill is placed and the relative size of the timbers and the large stones. The buoyancy of the timber depends, of course, on the kind of timber used. Frequently as much as 25 per cent of the entire volume of a rock-filled crib dam is composed of timber.

On account of the relative lightness of timber dams, the dimensions necessary to prevent sliding are almost always sufficient to prevent overturning. For stability against sliding, the effective weight of a rock-filled crib dam on a rock foundation, including the vertical water pressure on the upstream face, ranges from 2.5 times the horizontal pressure of the water, for unimportant, temporary structures on rough foundations, to 4 times the horizontal pressure, for important dams on smooth foundations.

The stability of A-frame and beaver dams, which have no rock fill to prevent sliding, depends almost entirely upon the strength of the pins which fasten the

dams to rock foundations, unless the foundation is so rough as to permit a horizontal support for the bottom timbers. Friction of wet timber on stone is very small.

The factors of safety to prevent sliding of timber dams on earth foundations follow closely those recommended for masonry dams on earth.

The timbers of the dam should be investigated for strength to transmit the loads to the foundations. In rock-filled dams, much of the load is transmitted through the rock fill, thus relieving the stress on the lower timbers.

6. Tightening the Foundation. If the dam rests on a rock foundation, the lagging at the upstream toe should be framed as closely as possible to the rock and the junction properly sealed. In some cases the rock is carefully cleaned and a layer of concrete deposited against the toe, as indicated in Fig. 1. In other instances, a fill of impervious earth is deposited against the upstream face of the dam, provided the velocity during floods is not sufficient to disturb it. Low dams on silt-laden streams may have a tight layer of sediment deposited against them during the first freshet.

Timber dams on earth foundations, without adequate sheet piling at the upstream toe, are precarious, even though an impervious fill is placed above the dam. Great care should be exercised to obtain a tight bond between the top of the piling and the lagging of the deck, and a splice plate of steel thoroughly fastened by lag screws is advisable, as a slight movement of the dam is likely to loosen the junction. It is also advisable, where sheet piling is used, to provide a vertical upstream face at least 4 or 5 ft high and allow the sheet piling to lap this face completely, in order to afford better opportunity for fastening it to the dam. This arrangement is shown in Fig. 4.

For further general information on foundation treatment refer to Chapter 3.

7. Protection Against Erosion. Spillway dams must be protected against erosion from the overflow, if the foundations are soft. This is usually accomplished by sloping or stepping the downstream face, as indicated in Fig. 4, and providing an apron to protect the foundations. The apron should be a low rock-filled crib with sufficient rock above the bottom timbers to prevent flotation; or, the apron may be anchored to round piling. A row of short piling and a fill of large rock fragments protect the lower end of the apron from being undermined, as shown in Fig. 4. (See also Chapter 3.)

8. Choice of Type. The beaver type of timber dam is the lowest in cost if plenty of timber is available. With more expensive timber, the A-frame type is usually the cheapest. Brush-topped beaver dams are seldom used for permanent structures.

The advantage of the beaver and A-frame types over the rock-filled type is found in their smaller first cost and lower maintenance charges. Rock-filled dams are hard to repair, as the timbers, being buried in the fill, are difficult to replace. The greatest objection to the A-frame type is its danger of failure when neglected. The rock-filled dam is in a large measure supported by the fill and will stand some time after the timbers have become materially decayed.

In the usually remote contingency of nearly complete submergence, which results in negligible head on the crest, the beaver and A-frame dams are likely to float. Neither of these types is easily constructed in deep water, while crib

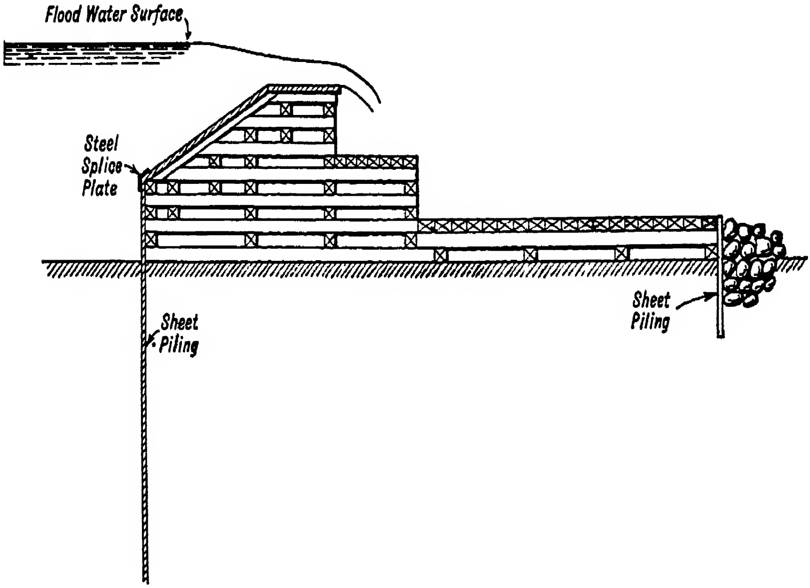

FIG. 4.

dams can be partly constructed on land, floated into place, and sunk by filling with stone.

The A-frame type is not particularly suited to earth foundations requiring sheet piling, as the desirability of a vertical upstream face for lapping the sheet piling, as previously described, reduces to a considerable degree the length of the sloping deck, on which the vertical water pressure is necessary for the stability of the dam.

9. Limitations of Timber Dams. Rock-filled timber dams have been constructed successfully to a height of about 70 ft; but very few dams of this type are higher than 20 ft. The beaver type is limited by the length of the trees available. A-frame dams higher than 20 ft are seldom encountered.

CHAPTER 23

DETAILS AND ACCESSORIES

1. Construction Joints. Construction joints in concrete dams are required to prevent haphazard cracking, which may prove unsightly and even dangerous. The best preventive for such cracking is a strict adherence to modern methods of control of temperatures of the concrete while setting and curing. This feature is outlined in Art. 14 of Chapter 15. However, the art of placing concrete has not been perfected to the extent that construction joints can be eliminated entirely.

Construction joints may be divided into three classes: (*a*) Horizontal joints, (*b*) transverse joints, and (*c*) longitudinal joints.

HORIZONTAL JOINTS

The height of horizontal joints or "construction lifts" is limited to the necessity of providing sufficient cooling between pours, since shrinkage due to temperature changes tends to form cracks.

For solid gravity dams, lifts of about 5 ft, with half that amount for the layer immediately on the rock, have become popular. Lifts of 10 or 15 ft for hollow and thin arch dams are permissible, depending upon thickness.

Modern methods for placing concrete and the treating of the top surface of each lift before the next lift is poured obviate the necessity of sloping the surfaces or providing keyways in solid gravity dams. However, for the thin, more highly stressed buttresses of hollow dams, sloping joints as shown in Fig. 1 for the Pensacola Dam or keyways as shown in Fig. 2 for the Possum Kingdom Dam have been used.

Modern treatment of the surface and good concrete create amply tight horizontal joints, and no provisions to insure watertightness, such as water stops, keyways, or other devices, are necessary even in thin arches or decks of hollow dams.

TRANSVERSE JOINTS

Transverse joints, i.e., joints normal to the axis of the dam, are necessary in order to prevent haphazard transverse cracks due to contraction of the concrete. In solid dams, such cracks are objectionable mainly from the standpoint of appearance. In arch dams, such cracking is objectionable on the ground of both leakage and danger, since the cracks, if allowed to occur, might not be normal to the line of stress when the dam becomes loaded. In hollow dams, joints between the buttresses and the deck are a construction necessity.

A study by Byram W. Steele [1] of the relation between cracking and influencing factors, including transverse joint spacing in 78 concrete dams, led him to the conclusion that, although the proper spacing of such joints depends upon a num-

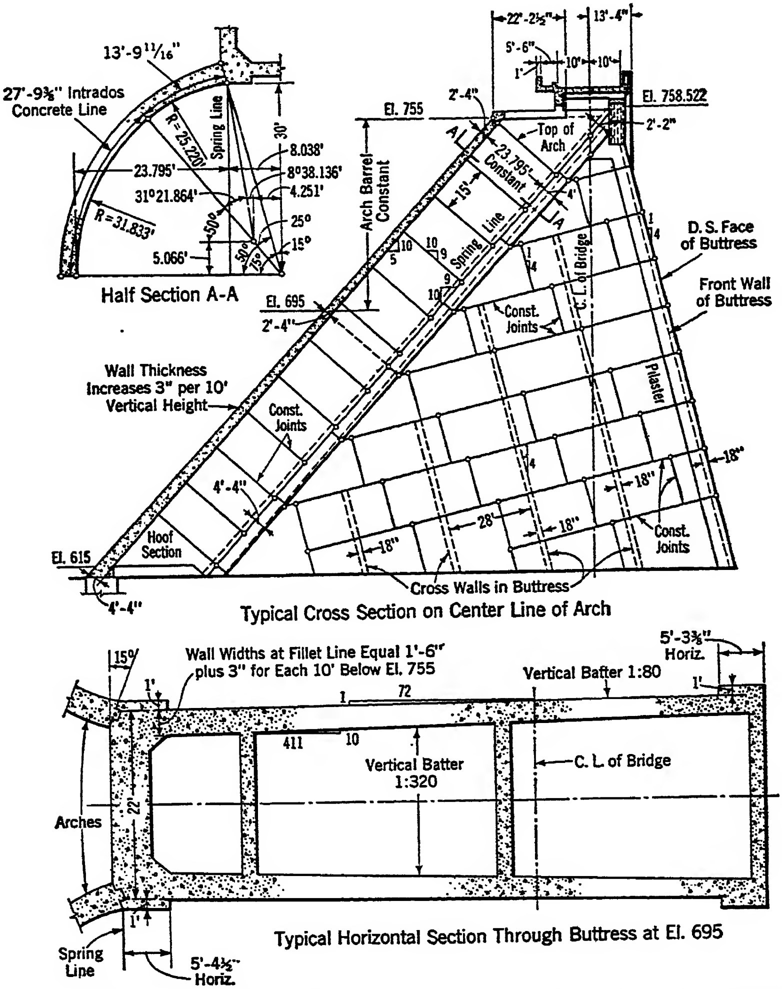

FIG. 1. Pensacola Dam, featuring inclined construction joints. (*M. G. Fuller, in Eng News-Record, Feb. 1, 1940, p. 42.*)

ber of governing conditions, there does not appear in available literature an acceptable set of rules governing this feature. He finds that, at present, a spacing of about 50 ft is common practice, such joints of course extending entirely

[1] "Construction Joints," *Proc. Am. Soc. Civil Engrs.*, May 1940, p. 908.

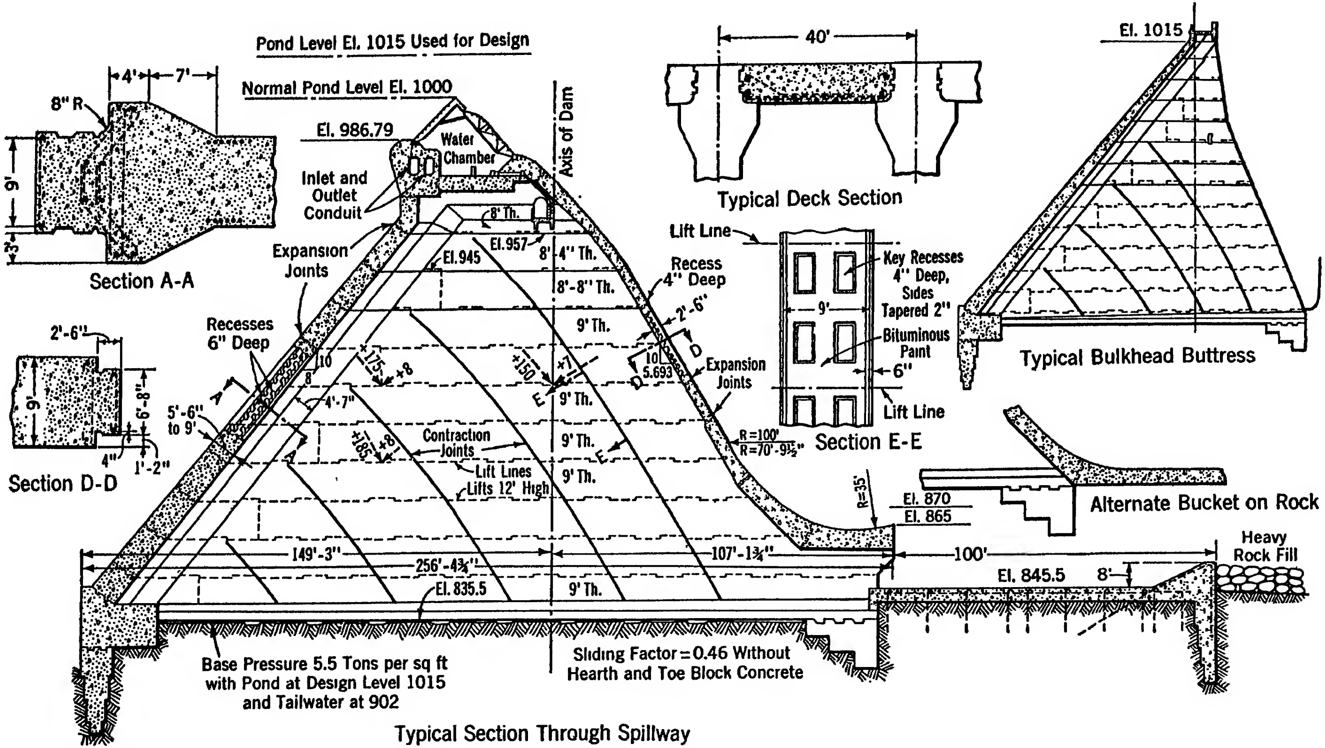

FIG. 2. Possum Kingdom Dam, featuring diagonal construction joints. (*From Eng. News-Record, June 8, 1939, p. 71.*)

through the structure. The joint spacing in the higher dams of this country investigated by Steele is shown in Table 1.

TABLE 1

SPACING OF JOINTS IN AMERICAN SOLID CONCRETE DAMS OVER 200 FEET HIGH

Dam	Height	Spacing in Feet: *Transverse*	Spacing in Feet: *Longitudinal*
Arrowrock	351	25, 50, 150 *	None
Ashokan	252	84, 168 *	None
Boulder	726	25 to 66 †	30 to 50 †
Bull Run	200	40	None
Conchas	235	40 to 50	30 to 50
Don Pedro	288	32.5, 65, 130 *	None
Elephant Butte	306	50, 100 *	None
Exchequer	330	25, 50, 75	None
Friant	300	50	None
Gibson	205	30, 60 *	None
Grand Coulee	540	50	50
Hiwassee	307	38 to 50 ‡	None
Kensico	307	73.5 to 79	None
Madden	223	56	None
Marshall Ford	265	52	42
Melones	210	..	None
Morris	328	50	None
Narrows	216	50	None
New Croton	297	None	None
Norris	265	56	None
O'Shaughnessy	344	97, 48.5 *	None
Owyhee	405	50	None
Pardee	358	37.5, 75, 150 *	None
Seminoe	290	50	None
Shasta	560	50	50
Shoshone	238	None	None
Tygart	240	52, 60	None
Waterville	200	50	None

* Closest at top of dam.
† Arched dam—smaller spacing at downstream toe.
‡ 38 ft in the 260-ft length of spillway.

Although no precedent has been established, it would seem that the spacing of transverse joints in arch and solid gravity dams should not exceed the height of the dam, with 50 ft as the maximum.

Dams covering faults are subject to bad cracking, should such faults become active in the slightest degree. This feature of course demands special consideration such as that used for the Morris Dam in California.[2]

[2] *Eng. News-Record*, December 27, 1934, p. 825.

In order to insure intimate contact for the transmittal of stress, transverse joints in arch dams must be consolidated either by grouting or by leaving a slot to be filled later after the monoliths have ceased shrinking.

The necessity for grouting transverse joints in solid gravity dams is distinctly open to question and is not always done.

Keyways of various types have been used in transverse joints of solid gravity dams. A typical example is indicated in Fig. 3. For low dams on earth where unequal settlement might rupture the transverse joint water stops, substantial

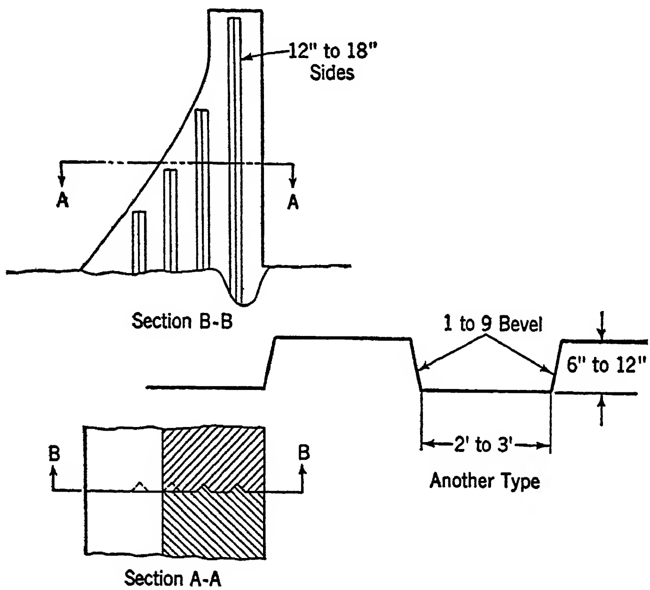

FIG. 3. Keyways used for transverse joints.

keyways have been used, but for dams on rock and particularly when high enough to provide heavy compressive stresses, it is essential that each monolith should be capable of functioning independently and any ties between monoliths, such as keyways, tend to prevent this.

There is no necessity for the application of any type of bituminous treatment of transverse joints, and few modern dams are thus treated.

Many low dams have been built with no water stops in the transverse joints to prevent leakage. Since the gradient and therefore the discharge per foot of height of dam is theoretically the same for high as for low dams, the necessity for water stops or any other provision for reducing transverse joint leakage is not particularly apparent.

However, most large modern dams are provided with such water stops unless

the joint is grouted, in which case, water stops or rather "grout stops" of a less permanent nature must be used to retain the grout.

The exceptions are in dams on earth, where leakage through the joint may endanger the stability of the foundation, or in dams where a good appearance of the downstream face must be preserved.

Water stops, drainage, and other provisions for preventing leakage through transverse joints are designed to no particular standard. They vary from the

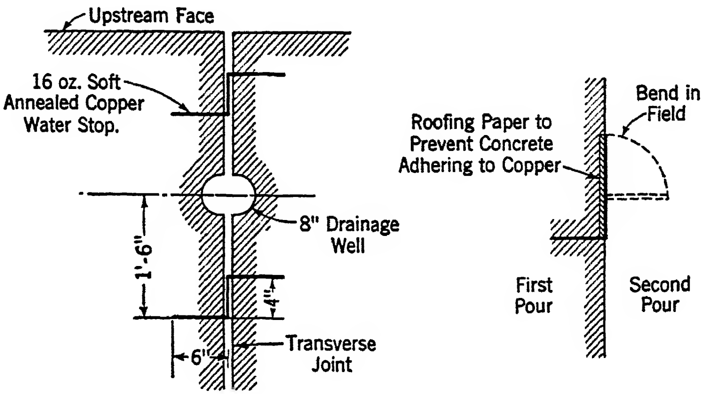

FIG. 4. Type of transverse joint seal.

use of a simple copper water stop to the provisions for water stops combined with wells to be filled with asphalt or other plastic material.

A typical arrangement is shown in Fig. 4, used by the U. S. Army Engineers in the construction of the Loyalhanna, Pa., Dam, in which are provided two ordinary copper water stops near the upstream side of the joint which confine an 8-in. drainage well. If desired, the drainage well can be filled later with melted asphalt or grouted.

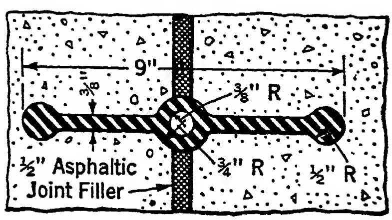

FIG. 5. Rubber seal used for Imperial Dam. (*From Eng. News-Record, Feb. 1, 1940, p. 48.*)

Water stops of this type can be used only for structures on foundations which are not expected to yield appreciably, as otherwise they will break. A yielding type made of rubber,[3] shown in Fig. 5, has been used for a number of dams. Actual displacements of 2 in. have been observed at the seals of the Imperial Dam without noticeable leakage. In locations which are always wet and dark such rubber seals are believed to be permanent. Under other conditions they should never be used.

[3] C. P. VETTER, "Rubber Waterstops for Dams," *Eng. News-Record*, February 1, 1940, p. 47.

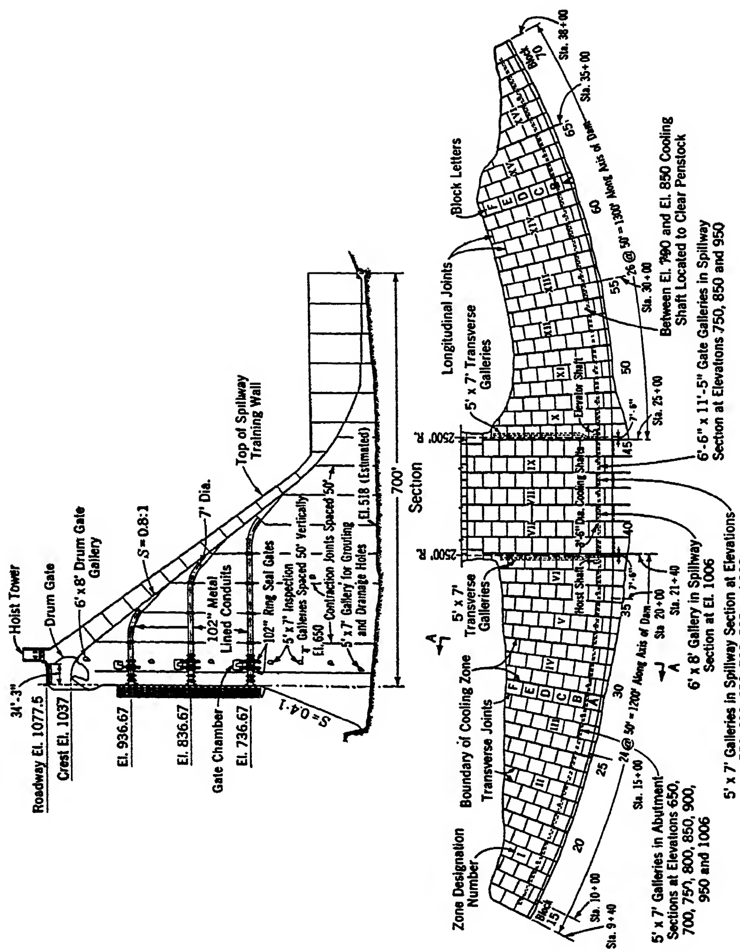

FIG. 6. Shasta Dam, showing construction joints. (*From Eng. News-Record, May 5, 1938, p. 649, and Western Construction News, May 1938 Supplement, Sheet No. 3, respectively.*)

As it is required that the United States Fish and Wildlife Service approve facilities for fish protection on all Federally built dams, and the fishery departments of many states have similar requirements respecting private enterprises,

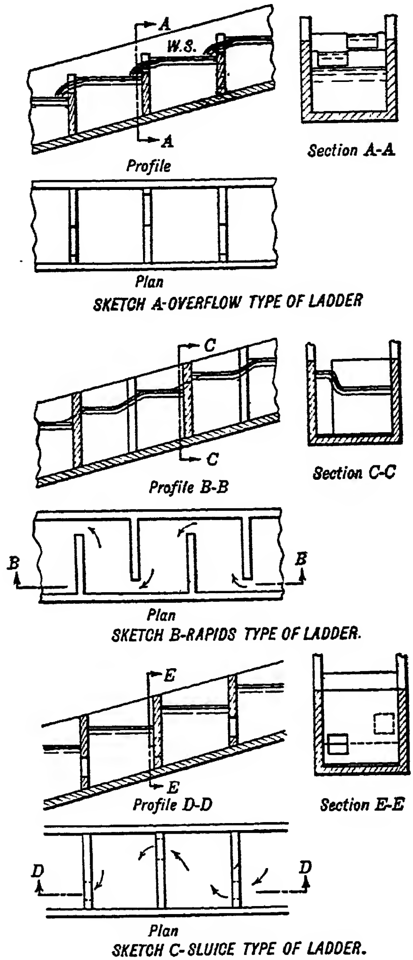

Fig. 15. Diagrammatic sketches of types of fish ladders.

these departments should be consulted at an early stage in the design of a dam. Suggestions and standards are always available. For this reason only general types will be described here.

In the design of fish ladders most commonly used, which is shown in sketch *A* of Fig. 15, the baffles are in the form of complete cross walls, or weirs, that divide the passage into a succession of rectangular pools. The water passes from one

pool to the next by flowing over these weirs. There is a difference of opinion among the fishery experts as to whether the weirs should have full length level crests or should have a rectangular notch, 6 in. to a foot deep, extending one-third to one-half the way across the passage. Among those who prefer the notch there again is a difference of opinion as to whether the notches in successive weirs should be in line down the center of the structure or staggered at opposite ends of successive weirs.

The dimensions of a fish ladder of this type should be influenced by operating conditions, such as the size of the stream, height of the dam, location and nature of fishway entrance in relation to other parts of the structure, size of fish, and the magnitude of the run. At the Bonneville Dam on the Columbia River, where all conditions indicated the necessity of large fishways, the fish ladders were made 40 ft wide, with baffles spaced 16 ft apart, and with water in the pools 6 ft deep. On smaller streams, fish ladders as small as 6 ft in width, with baffles 4 ft apart and a water depth of 3 ft are giving satisfactory service. Even smaller pools are satisfactory for the use of small fish at low dams, but the 6- by 4- by 3-ft pool is considered to be the minimum for salmon and other large fish.

It is a common layman's belief that some species, such as salmon, prefer to jump, but experience in fishway operation has shown that all species resort to jumping only when their passage by swimming appears to be blocked. The height to which a fish can jump depends to a great extent upon its ability to gain momentum before leaving the water. The confined space in the pools of a fish ladder offers little opportunity for gaining momentum for jumping from one pool to the next. Jumping in a ladder is considered objectionable, for the reason that a large proportion of the jumps are poorly directed, with the result that fish frequently strike the side walls or jump clear out of the ladder. It therefore is preferred that the difference in elevation between successive pools of the ladder and the depth of water flowing over the weirs be such that the fish are induced to swim rather than jump from one pool to the next. This condition usually is fulfilled if the depth of water flowing over the weirs is equal to the difference in elevation between successive pools.

Owing to the necessity of dissipating the velocity in each pool of the ladder, the quantity of water that can be allowed to flow through it and, accordingly, the difference in elevation between successive pools, are limited. A difference of 1 ft in elevation between pools is most commonly recommended; 15 in. is stated to be the preferred maximum for small ladders. Differences as great as 2 to 2½ ft have proved satisfactory where the ladder consists of only a few steps and the pools are adequately large and deep.

Many other arrangements of baffles have been proposed and used. Sketch *B* of Fig. 15 is the rapids type, in which the baffles are shorter than the width of the trough, leaving openings at the end through which the water passes. The openings are staggered, as shown, so that the water takes a circuitous path, in order to kill as much velocity as possible.

In the British Isles the preference is for complete partitions between pools, with submerged orifices for the passage of water and fish, as shown in sketch *C*

of Fig. 15. One of the principal advantages claimed for this type is that it minimizes the entrapping of air bubbles in the water. This design also assures a route by which the fish can swim from one pool to the next. It has the further advantage of maintaining a uniform flow with fluctuations in headwater level.

Another arrangement of baffles, known as the "Denil" design, has been used successfully in Europe. In this design closely spaced baffles placed along the sides and bottom of the passage are shaped to form curved pockets. A part of the water flowing down the central, unobstructed portion of the passage enters these pockets and is directed abruptly backward into the central area, where it

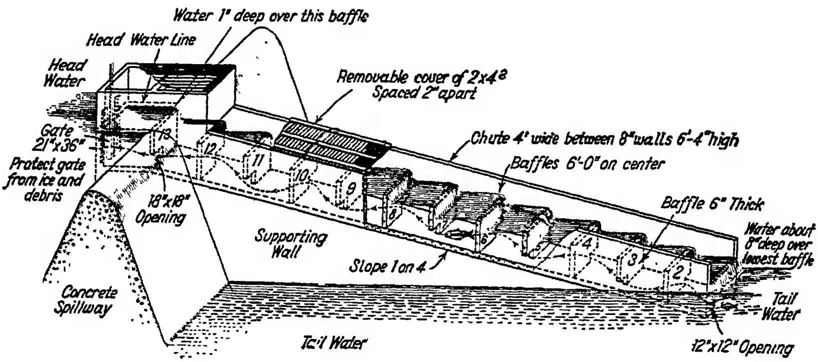

FIG. 16. View of reinforced concrete fish chute. Reinforcing not shown. *State of Michigan, Department of Conservation.* Make openings through baffles increase in area uniformly between the lowest and highest opening.

impinges against the main body of water, thereby checking its velocity. This arrangement has been found to be an unusually efficient energy dissipator. A fishway of this design accordingly can have a much steeper slope than the pool type ladder without developing excessive water velocity. It is claimed that this design also is less affected by fluctuations in headwater elevation. The Denil type of fishway has not been used in America, but experiments recently conducted at the Iowa Institute of Hydraulic Research [8] have shown pleasing results for this type in comparison with others tested.

In contrast to fish ladders, in which the fish climb from the lower to the higher pool level by their own efforts, many mechanical devices have been proposed and used for lifting fish over obstructions. One of the most successful of these is the fish lock, which incorporates the essential features of a navigation lock. That is, it consists of a lock chamber, a gate-controlled entrance by which the fish enter the chamber at the lower level, a similar means of permitting the fish to leave the chamber at the higher level, and a system of valves for alternately filling and draining the chamber. A simple device of this kind, known as the "Barr" fish

[8] See A. M. McLeod and Paul Nemenyi, "An Investigation of Fishways," and Paul Nemenyi, "An Annotated Bibliography of Fishways," University of Iowa Studies in Eng. Bulls. 24 and 23, respectively, 1941.

lock, has been successfully applied in a number of places in the Middle West. Fish locks of more elaborate design were included in the extensive system of fishways built at the Bonneville Dam in the Columbia River and have been found to function satisfactorily. At the Concrete Dam on the Baker River, in Washington, fish are being lifted successfully over the dam in a tank of water.

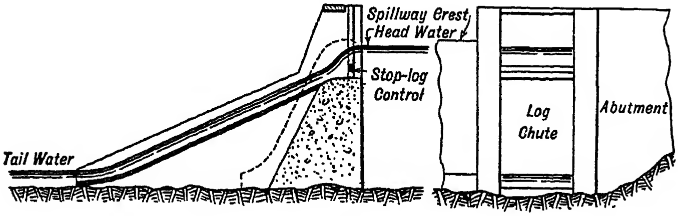

FIG. 17. A typical log chute.

One of the most important features in the design of any fishway is the nature and position of its entrance, whether it be a fish ladder, fish locks, or a mechanical device. The designer should keep in mind the fact that the fish, in their efforts to proceed upstream, will follow the prevailing flow below the dam. In finding

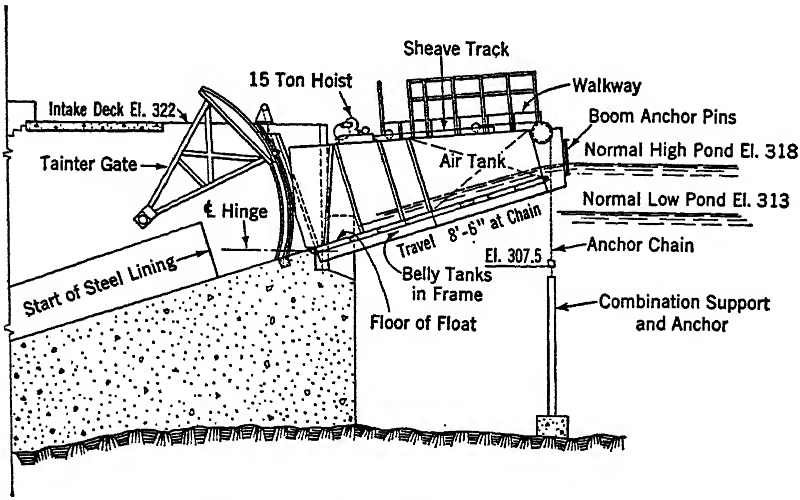

FIG. 18. Log sluice for Williams Station, Central Maine Power Co.

their further progress blocked by the dam, they will seek a route of passage, where the principal flow seems to give greatest promise of success. The entrance to a fishway therefore must be placed near the base of the obstruction, where it can compete most favorably with the impassable flow in attracting the fish. Each dam presents its individual problems in this regard. The selection of the site for the fishway must be based upon a careful study of flow conditions. The high

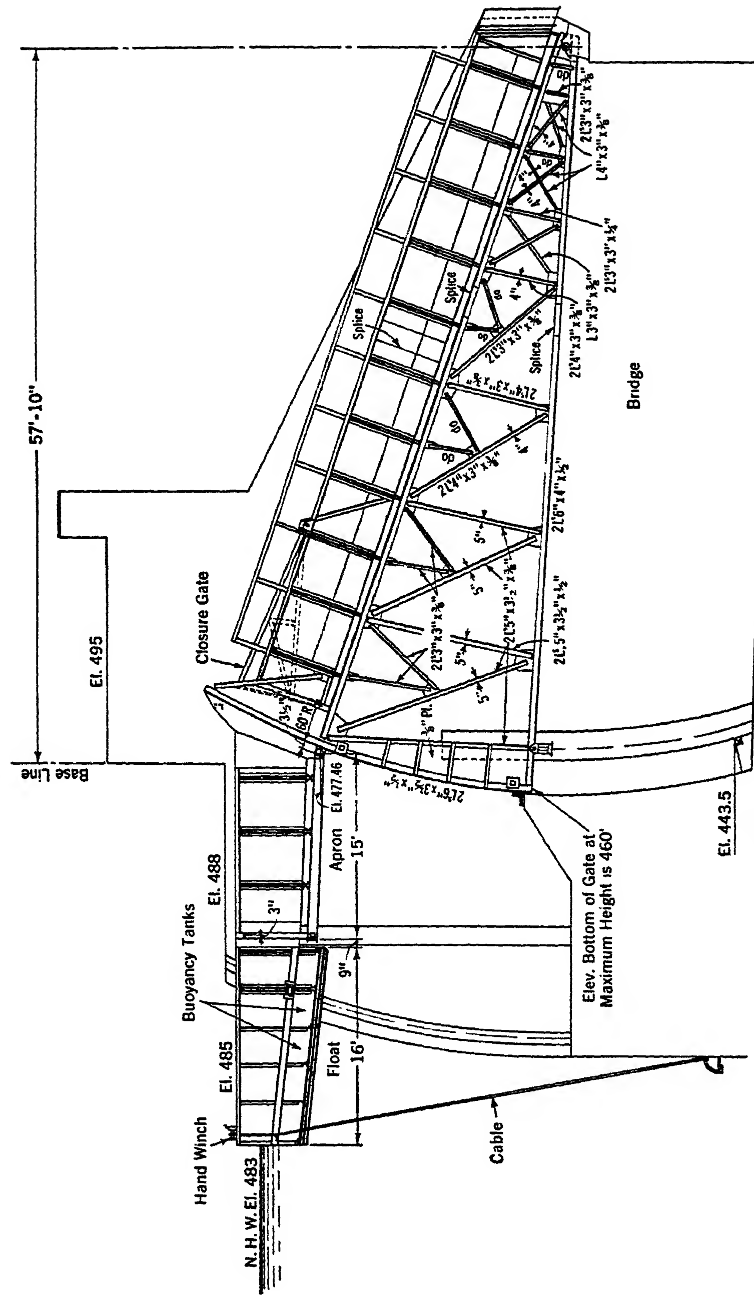

Fig. 19. Log sluice for Wyman Station, Central Maine Power Co.

velocities below a spillway frequently present a barrier that forces the fish toward the two shore lines. The reluctance of the fish to cross such a barrier usually makes it necessary to provide fishways at each end of the dam.

4. Log Chutes. Log chutes may be required by law in places where there are logging operations on the stream. A typical chute is shown in Fig. 17. It consists of a depression in the crest of the dam, having a bottom elevation 2 to 4 ft below low-water surface and provided with a control through which the water passes to a smooth inclined flume. The logs may then pass down this flume to the lower pool. The gradual incline is necessary to prevent the logs from im-

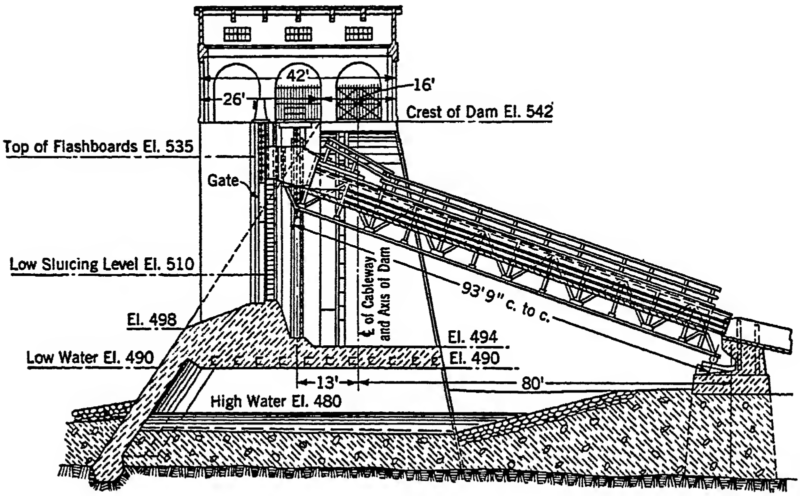

FIG. 20. Log chute at Aziscohos Dam. (*Eng. News, Vol. 65, No. 10, p. 290.*)

pinging at high velocity directly on the foundation of a shallow lower pool, with danger of their being split. The inclination of the chute varies greatly with local conditions.

The control, which regulates the flow through the chute, is usually of a type that lowers when more water is required. Drum gates, bear-trap dams, and stop logs, all described hereinafter, are frequently used for this purpose.

The chute should be made as narrow as possible in order to provide the required depth for passing logs with a minimum discharge. The depth of the water passing the control can be regulated by the operator; but the width is usually not adjustable. The width required depends, of course, upon the size of the logs and the rate at which they must pass the dam. While some log chutes have been provided to pass only one log at a time, chutes 30 ft wide have also been built for enormous logging operations.

The discharge required to drive logs is sometimes a serious drain on the flow available for other uses, principally for those streams which are almost completely regulated. The discharge necessary to float the logs past rapids in the river may

be far in excess of that required to get them by the dam. In such cases the chute must have sufficient capacity for that purpose, unless other means are provided to draw water from the pond.

Timber booms are necessary to guide the logs head on toward the entrance of the chute. For large fluctuations of headwater surface, very elaborate controls are necessary. For some such cases, the chute is pivoted at the lower end, and the upper end is raised and lowered according to the elevation of the water surface in the pool.

Fig. 18 shows a log chute designed for a fluctuation of 8.5 ft in headwater. The chute is floated by means of air tanks, which are depressed to the amount necessary by pulling on the anchor chain with the 15 ton hoist. It is 15 ft wide at the upstream extremity and 6 ft at the entrance to the sluice proper.

Fig. 19 shows a log chute designed for a fluctuation of 16.5 ft. The chute is placed on the upper member of a tainter gate, which is lowered as the pond is drawn down. The chute is 6 ft wide. Above the chute is a floating apron which flares from 6 to 20 ft in width.

The largest chute, to the author's knowledge, is that built for the Aziscohos Dam,[9] to acommodate a fluctuation of 25 ft. This chute is shown in Fig. 20. Its operation is apparent. It has a 7-ft chute with a 24-ft entrance mouth.

[9] *Eng. News*, Vol. 65, No. 10, p. 289.

CHAPTER 24

HEADWATER CONTROL

Revised by Chas. M. Wellons [1]

1. Spillways and Sluices. In any dam that serves to impound the runoff from rainfall, some means must be provided to pass that part of the inflow to the reservoir that cannot be stored or used immediately. Spillways and sluices are commonly used for this purpose. A spillway is a surface over which water from the surface of the reservoir falls to the tailwater level. A sluice is a passage through the dam by which water may be drawn from a deeper level of the reservoir space. Spillways are ordinarily designed to accommodate large volumes of flow. Sluices are ordinarily designed to pass relatively small volumes in order to draw the surface of the reservoir below the spillway crest level or to maintain a closely regulated outflow. Various considerations of design, operation, maintenance, and economy have given rise to numerous devices for use in the control of the discharge over spillways or through sluices.

It is impossible to describe within a limited space all the many devices that are used for the control of spillways and sluices or to define all the circumstances that affect their selection or operation. Practically all of these devices fall under or are related to the classifications given in the following text. Certain types of devices have been developed to meet particular circumstances or requirements. In the usual case, the choice between two or more types of control is likely to be based upon rather fine distinctions between their relative merits or upon personal preference. In general the success of the installation depends more on the design of details than on the type of control.

2. Spillway Control. In the simplest form of spillway, the discharge is regulated only by the amount of rise of the reservoir surface above the level of the crest of the spillway. If the crest is long enough relative to the required discharge capacity, the rise required may be unimportant. If, however, the length of the crest is limited, its level must be lowered so that the maximum expected flood can be discharged over the crest without the surface of the reservoir rising to where it will encroach upon properties or rights of others or exceed other prescribed limits.

The space between the level of the crest and the level of the maximum expected flood usually represents storage and head that can be used profitably. The main structure of the dam must be built to retain the reservoir at the higher level, and ownership of the land inundated or rights to flood it must be obtained. Use of the space above the crest therefore can be obtained usually at relatively

[1] Formerly Principal Engineer, U. S. Engineer Office, Pittsburgh, Pa.

small additional cost by the addition of suitable means for regulating the discharge over the spillway crest.

3. Spillway Control Devices. The usual devices for control of spillway discharge may be classified as follows:

1. *Crest control:*
   a. Temporary flashboards.
   b. Permanent flashboards.
   c. Drum gates.
   d. Bear-trap dams.
   e. Tilting gates.
2. *Crest gates:*
   a. Plain sliding gates.
   b. Roller-bearing gates.
   c. Fixed-roller gates.
   d. Truck-mounted gates.
   e. Taintor gates.
   f. Rolling gates.
   g. Stop logs and needles.
3. *Siphon spillways.*

As mentioned before, it is impossible to state all the factors that should be considered in selecting the type of control for a particular installation. The usual considerations are: the accuracy of regulation of the reservoir level required, the importance of the loss of water through leakage and operation, climatic conditions, the passage of ice and large drift during floods, the frequency and speed of operation required, the depth of the controlled pond and the maximum flood level above the crest, and the proportions of the controlled openings. These factors must be considered in connection with the cost of the operable installation, together with its related masonry and mechanical features and the cost of attendance, operation, and maintenance.

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4. Crest Control. The term "crest control" is used to include those types of controlling devices which raise or lower the effective crest level, thus regulating the volume of flow over their top surfaces or edges. These devices generally make use of the headwater pressure to raise or lower the damming structure and utilize relatively small differences in elevation of headwater to automatically regulate discharge.

(*a*) *Temporary flashboards.* Flashboards consist of a series of vertical boards or panels placed on the crest of the dam for the purpose of raising the pond level. Fig. 1 shows a typical system of temporary flashboards, consisting of a series of panels supported by pins or pipes which are inserted loosely into sockets set in the masonry crest of the dam. The pins or pipes are designed to bend over and release the flashboards when the water surface in the pond reaches a certain elevation above the top of the flashboards, thus automatically lowering the crest to pass excessive floods. In this country this is the most common of all devices designed to control the elevation of flood-water surface. The boards or panels

are fastened loosely to the supports and are lost when the supports bend over, unless removed before anticipated high water.

In order to facilitate the handling of a barge for removing and restoring the flashboards, it may sometimes be found desirable to provide, at intervals, sockets into which mooring pins may be set, as indicated in Fig. 1. Cableways may also be used as an anchorage for this barge. The boards and supports are sometimes manipulated from an overhead trolley or bridge on the crest of the dam.

Where it is possible to remove the flashboards in advance of floods, they are usually built in panels, as indicated, and provided with handles. For the type shown, handles are not permissible if considerable drift is anticipated, as this may

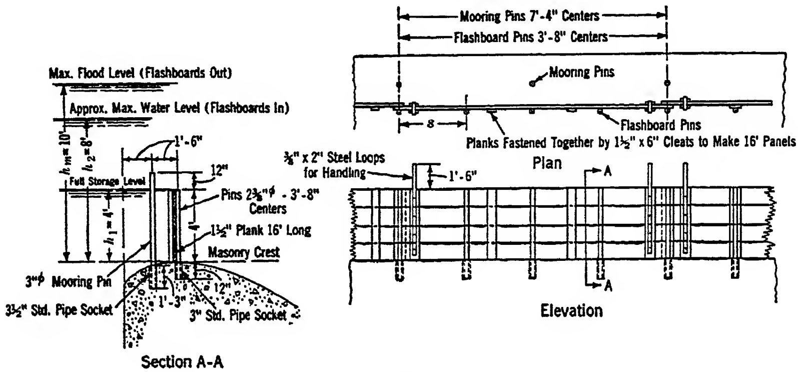


FIG. 1. Typical flashboard installations.

collect and cause premature failure. The boards usually have unplaned edges, and ashes or similar caulking materials are used to make them tight.

Flashboard pins are expected to carry a certain predetermined load and to fail when that load is exceeded. In commercial rolled steel bars, the elastic limit, ultimate strength, and behavior when the elastic limit is exceeded are quite variable, and the load under which failure will occur cannot be determined with any degree of exactness. A closer limitation of the range of load under which failure will occur is obtained by turning a circumferential notch at a point that will come immediately above the socket. Pins made of standard weight steel pipe will fail abruptly when overloaded and are therefore preferable to solid bars.

In some cases, the pipes are notched in order that they will snap off and pass with the flood, leaving the crest entirely unobstructed. However, this is not considered essential.

Some means is needed to hold the flashboards on the pins until they are loaded by the water pressure. This is usually done simply by nails driven into the boards and bent around the pins.

Tests on different sizes of galvanized steel and wrought iron pipe made by the Bureau of Standards in cooperation with the U. S. Forest Service [2] indicate that

[2] C. A. BETTS, "Controlling Flashboard Drop by Collapsing Pipe Supports," *Eng. News-Record*, April 30, 1936, p. 627.

the average stress [3] at failure was between 67,000 and 81,000 lb per sq in. with an average of about 75,000 lb per sq in. The greater strengths were found for the smaller pipe.

Table 1 shows stresses calculated from observed loadings on flashboard pins in actual service. In these cases, there was an undetermined amount of vacuum on the downstream side of the boards. The effect of this vacuum upon the loading was not included in the calculations. This accounts for the relatively low stresses indicated. In actual installation, considerable load that cannot be predicted with certainty acts on the pins because of the vacuum which usually occurs under the jet of the spilling water.

TABLE 1

Location of Tests	*Stresses in Pins at Failure Neglecting Vacuum (lb per sq in.)*
1. Sewalls Island	52,000
2. " "	43,000
3. " "	51,000
4. " "	56,000
5. " "	53,000
6. " "	52,000
7. " "	42,000
8. " "	43,000
9. " "	41,000
10. " "	58,000
11. " "	49,000
12. " "	48,000
13. " "	51,000
14. Connecticut River	55,000
15. Bonny Eagle	51,000
16. Ocoee No. 1	52,000
17. Mahoney	57,000

It is recommended that flashboard pins be designed for an initial stress of about 20,000 lb per sq in. with the water surface at the level of the top of the flashboards where they are subject to the pressure of waves and debris and ice, and therefore liable to premature failure. However, if they are well protected, the use of an initial stress of 30,000 lb per sq in. would be permissible.

Failure will occur when the final stress due to rise of water surface and vacuum under the jet becomes about 75,000 lb per sq in. The amount of vacuum is indeterminate, but Table 1, which was compiled from data on flashboards between 2 and 4 ft high, with 1 to 2.5 ft of water passing over them at failure, indicates that a design based on a stress of 50,000 lb per sq in. at failure would approximately take care of the force of vacuum if it is neglected in the calculations.

For an example, let us assume that it is possible to raise the water surface during floods to an elevation 100.0, a higher elevation of water surface flooding lands not owned.

[3] Based on Straight-line formula.

Assume that a 6.5-ft depth is required to pass the maximum flood over the spillway after the flashboards have failed. Then the permanent spillway crest would be placed at elevation 93.5.

With these data, it is desired to design flashboards which will fulfill the following conditions: (1) they must be as high as possible in order to provide the highest elevation of normal water surface, (2) they must fail before the water surface rises above elevation 100.0, or 6.5 ft above the permanent dam crest, (3) they must not fail so frequently under medium floods as to be uneconomical, (4) they must be reasonably safe from premature failure under impact of waves and debris.

Assuming that the flashboards are fairly well protected from the forces of waves and debris, an initial stress of 25,000 lb per sq in. is permissible.

Adopt, tentatively, a 3.9-ft height of flashboards, leaving a margin above them of 6.5 minus 3.9, or 2.6 ft. In Fig. 2, at the left-hand margin, find $H = 3.9$ ft. Trace horizontally to intersect $h = 0.0$ ft, corresponding to water surface at the crest of the boards, thence vertically to intersect the initial stress K, of 25,000 lb per sq in., thence horizontally to intersect the desired spacing of pins, say $d = 3.5$ ft, thence vertically to the lower margin and find a required section modulus S of 1.1 and a standard 2½-in. pipe pin.

Reversing this procedure, we find that for $S = 1.1$, a spacing of 3.5 ft and a final stress K of 75,000 lb per sq in., the water surface would be $h = 2.6$ ft above the top of the $H = 3.9$ ft flashboards when failure occurs, or at $H + h = 3.9 + 2.6 = 6.5$ ft, which is desired, and our tentatively adopted height of boards is correct.

The foregoing is on the assumption of no vacuum under the jet. It has been previously shown that the effect of a vacuum may reduce the indicated final stress to 50,000 lb per sq in.

Therefore, repeating the procedure for a final stress of 50,000 lb per sq in., we find that failure may occur at a head, h, on the flashboards of 1.3 ft.

With this design, we are reasonably sure that failure will occur with between 1.3 and 2.6 ft above the flashboards, with the probable value closer to 1.3 ft.

If it is felt that a flow corresponding to 1.3 ft over the boards will occur so frequently as to make the failure of the flashboards a nuisance, or too great an expense, then the only alternative is to reduce the height of the boards and also the initial stress in the pins.

For instance, the use of a 3-ft height of flashboards of the same size but with a 5-ft spacing and with an initial stress of 16,000 lb per sq in. instead of 25,000 would result in their going out with between 2 and 3.5 ft over them. Thus, the elevation 100.0 will not be exceeded in the event that the vacuum load is not created, and the head, h, at probable failure has been increased from 1.3 to 2.0 ft with greater flood-carrying capacity before flashboard failure, but at the sacrifice of a lowering of the normal water surface 3.9 minus 3.0, or 0.9 ft.

The foregoing theory applies to standard dam crests. Unpublished experiments by Hibbert Hill have shown that, in the case of flat-crested weirs, the deflection of the impinging jet on the horizontal surface at the elevation of the base of the flashboards will set up a dynamic reaction which will raise the water

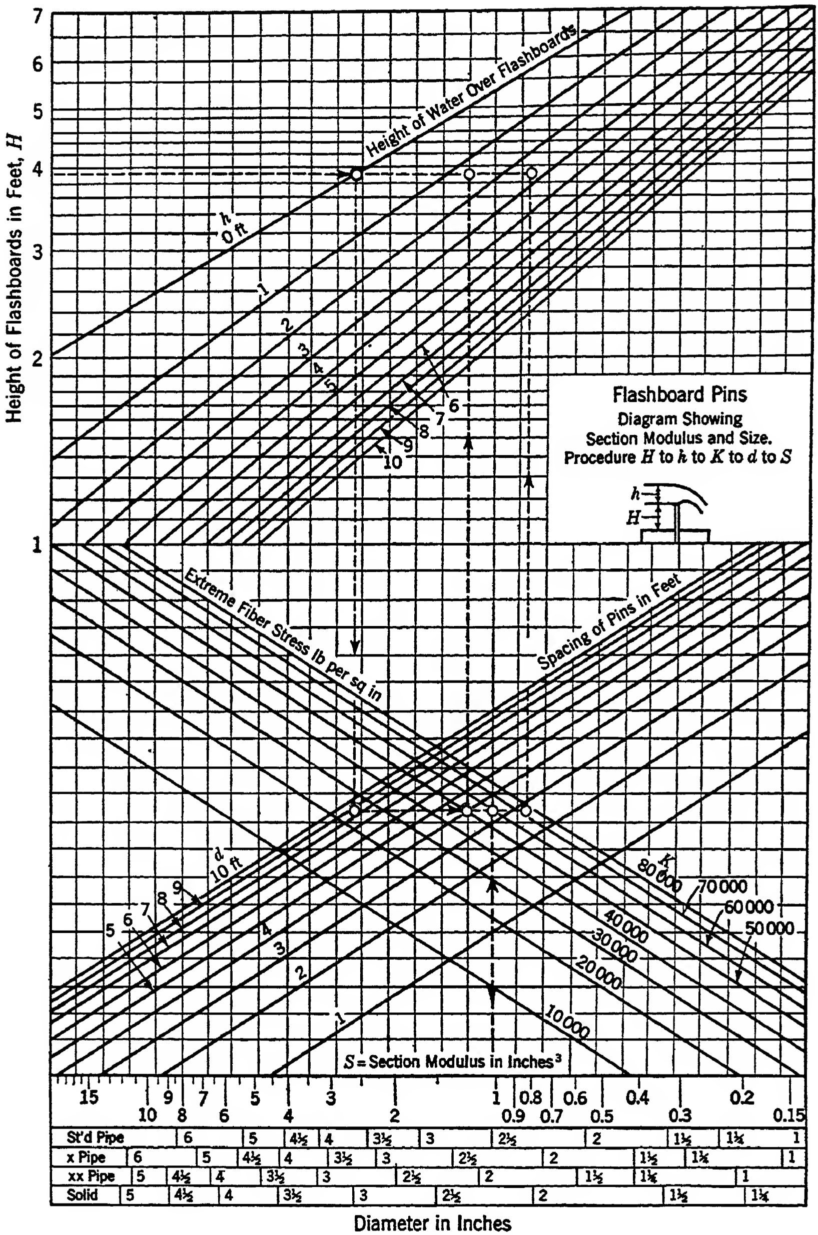


FIG. 2. Flashboard pins, diagram showing section modulus and size.

surface under the jet and materially reduce the calculated moment on the pins, as shown in Fig. 3, for values h/H greater than about $\frac{1}{3}$.

In some cases, the panels have been arranged as shown in Fig. 4. When the support, A, is removed, the panels revolve progressively around the pins until all the panels are either swept away or left hanging to the pins as shown by the dotted lines. This scheme leaves the pins in place and they can be used over again. However, if the river is subjected to a considerable run of debris during floods, enough of this may accumulate on the pins after the panels have gone to obstruct the crest without bending the pins, thereby causing higher water during floods than has been anticipated.

Temporary flashboards are always advantageous to increase the head for power at any dam not otherwise equipped with crest control, provided that the pond does not lap the development next above. Pipe sockets should be provided

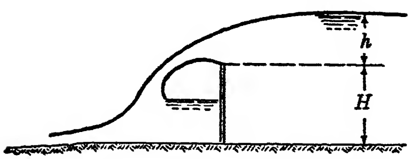


FIG. 3. Effect of vacuum behind flashboards.

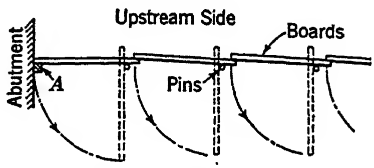


FIG. 4. Plan of flashboards designed for progressive failure.

in all crests of such installations, whether flashboards are contemplated in the near future or not. Temporary flashboards have been used up to a height of 10 ft.

After the boards are removed by a flood, the operators are usually able to replace them before the flow is reduced to the capacity of the turbines and the pond refilled by the tail end of the flood. With sufficient capacity of sluice gates or other emergency outlets, the water surface can be lowered to the elevation of the permanent crest before the flood has entirely receded, and the flashboards can be easily replaced. The sluice gates are then closed and the pond is filled up to the top of the boards.

The thrust of ice and, when the pond fluctuates, the lifting force of adhering ice frequently causes premature failure of flashboards. It is therefore necessary to maintain a channel of open water along the upstream face of the boards. This may be accomplished [4] (1) by cutting the channel by hand or with steam jets or ice saws, (2) by use of a bubbler system consisting of the release of jets of air at a lower elevation to bring warm water to the surface, (3) by use of heaters under covers stretched between the ice sheet and the flashboards.

(*b*) *Permanent flashboards.* Permanent flashboards are similar in principle to the temporary type, except that they are designed to operate automatically or by manipulation, without damage to themselves. They have been used mostly for special conditions and large installations.

[4] Fuller description in "Ice Problems in Hydro-electric Power Plants," *Nat. Elec. Light Assoc. Pub.* 082, August 1930.

Fig. 5 shows details of the permanent flashboards used at the Davis Bridge Dam of the New England Power Co. The supports are held in place by a seat on the crest at B and a latch on the bridge at A. They are removed during floods by tripping the latch. The boards, which are in the form of stop logs, are lost, and the supports are drawn up to the bridge by means of the chains and returned to place after the flood by fastening the top to the latch and swinging them down into the seat at the bottom. The new boards are then forced down

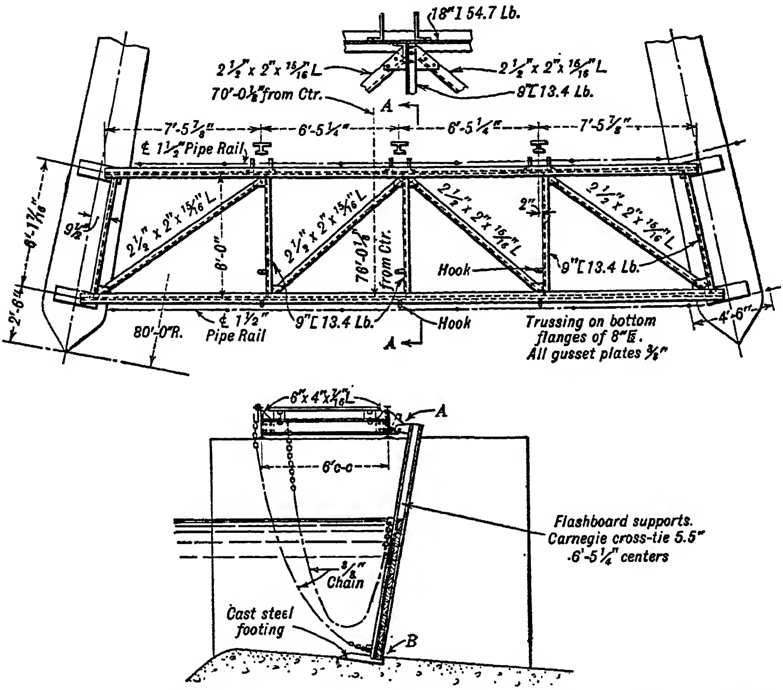


FIG. 5. Details of flashboard device, Davis Bridge spillway. (*E. A. Dow at spring meeting, Am. Soc. Mech. Engrs., 1925.*)

from the top. The piers shown in Fig. 5 are not parallel because the Davis Bridge spillway is curved.

One reason for using this type of flashboard in preference to the less expensive temporary type previously described is that the Davis Bridge spillway controls an extremely large reservoir. The tail end of the flood would not be of sufficient duration to refill such a large reservoir if the water surface were allowed to drop to an elevation close to that of the permanent crest in order to replace the flashboards, there being no other outlet of large capacity to be used to discharge the tail end of the flood while the boards are being replaced.

Fig. 6 is an illustration of a hinged leaf type of flashboard made up of panels which can be raised or lowered from an overhead cableway. The panels are

supported in position by a wooden strut. In an emergency, the panels can be dropped by drawing on a knotted cable which has been previously threaded through holes in the middle of each strut. As the cable is drawn in, the knot at the end engages each strut successively, breaking and removing the strut and allowing the panel to fall flat on the crest. The panel is said to fall uninjured, being well cushioned by air and water. The panels can later be raised again from the overhead cableway and new supporting struts installed.

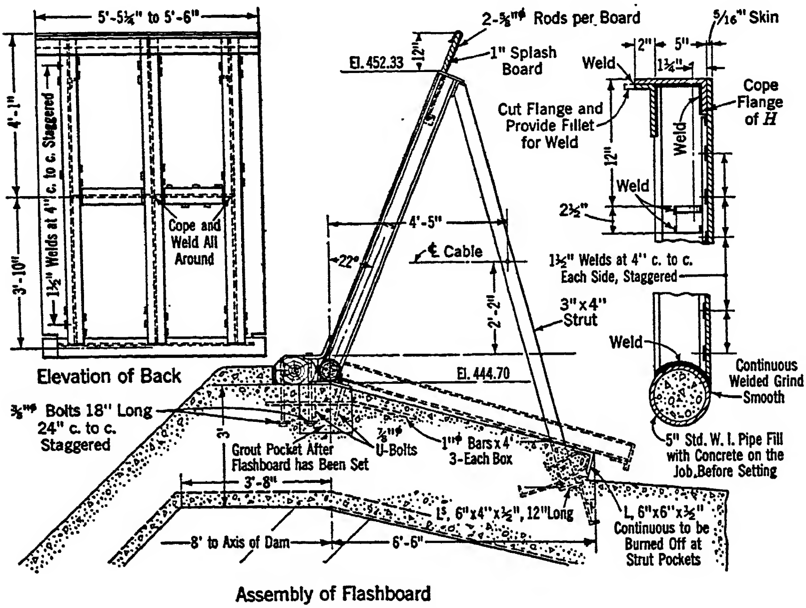


FIG. 6. Assembly of flashboard. (*From Eng. News-Record, Aug. 17, 1933.*)

Fig. 7 shows a flashboard device of a more permanent nature. It may be regarded as a part of the permanent structure that will be carried away by a flood that is greater than the regular spillway is designed to handle. The concrete slab structure is braced against the water load by steel struts that bear against a steel rail embedded in the concrete of the slab. The parts are arranged so that a rise of the pond above a predetermined level will produce a moment of sufficient magnitude to overturn the slab. The points of support are arranged so that panels will go out successively as the water level rises through a narrow range above elevation 760. A device of this kind is limited in its usefulness to auxiliary spillways that are provided for safety but not for regular operation. The effect of waves or ice at probable reservoir levels should be considered in connection with such a device.

(*c*) *Drum gates.* Fig. 8 shows the Stickney type of drum gate used for the New York State Barge Canal. Its crest is lowered automatically by rising headwater. The leaves are so proportioned that headwater in Chamber A sup-

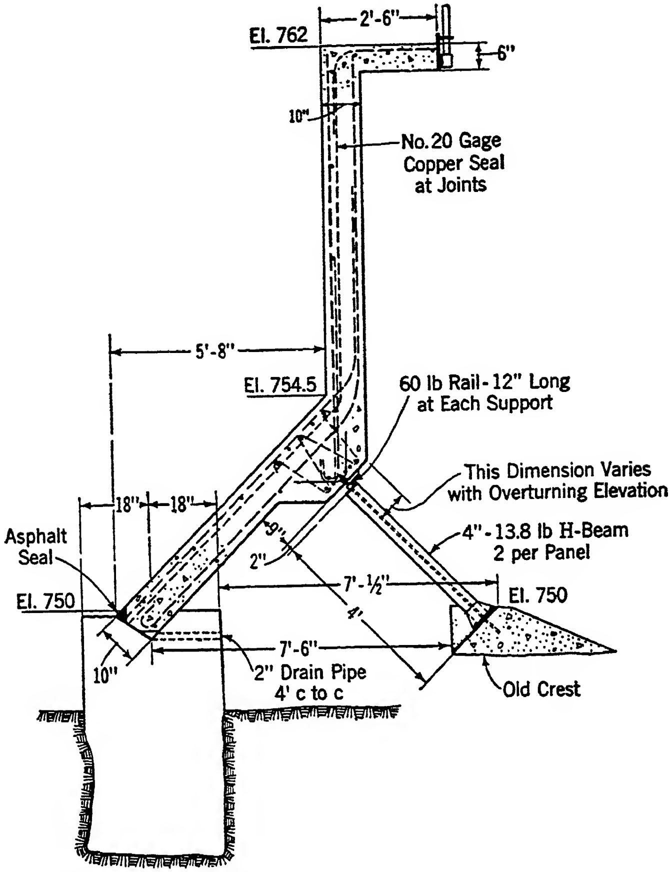


Fig. 7. Tilting section operates as emergency spillway. (*Eng. News-Record, Aug. 14, 1930 p. 263.*)

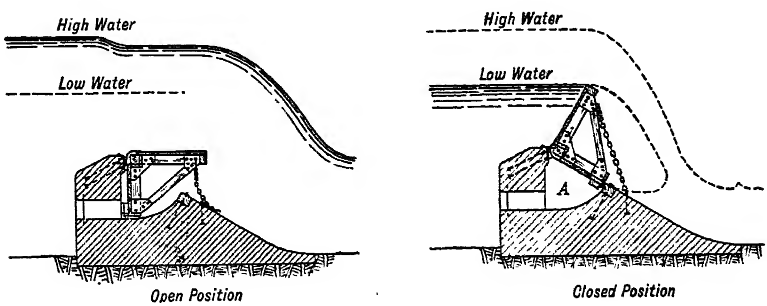
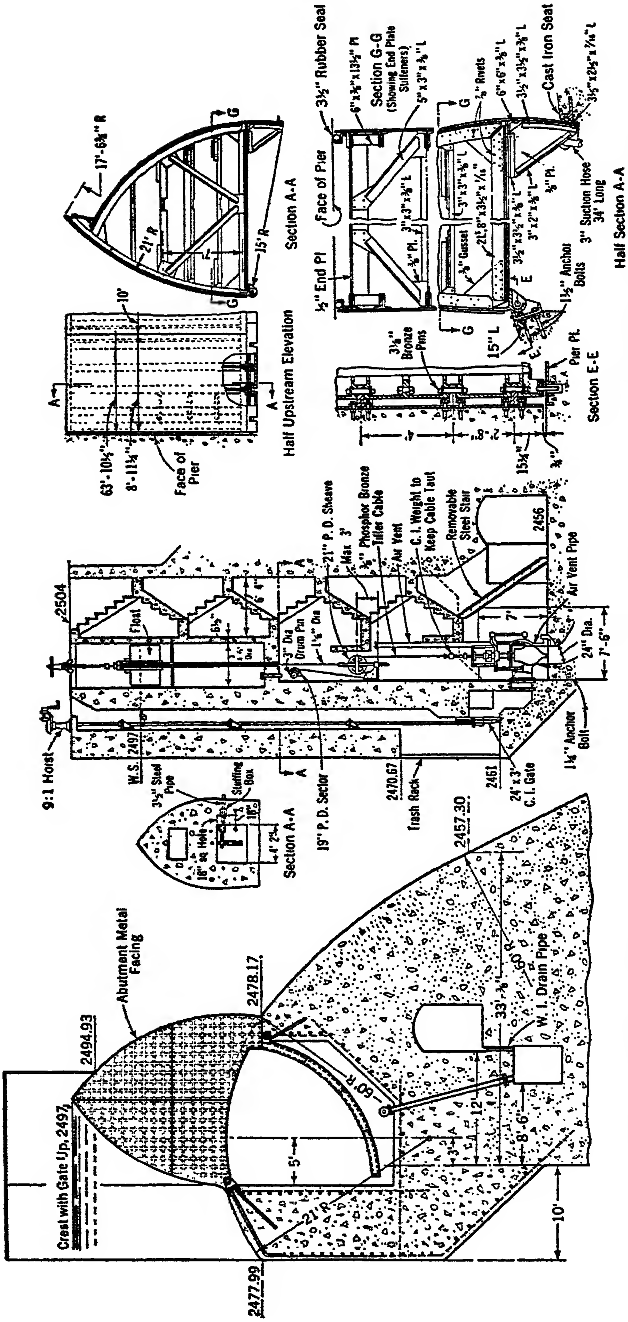


Fig. 8. Stickney type of drum gate.



(a) Enlarged Crest Section (Gate Removed)

(b) Arrangement of Operating Machinery

(c) Details of 14½ x 64 ft Drum Gate

FIG. 9.

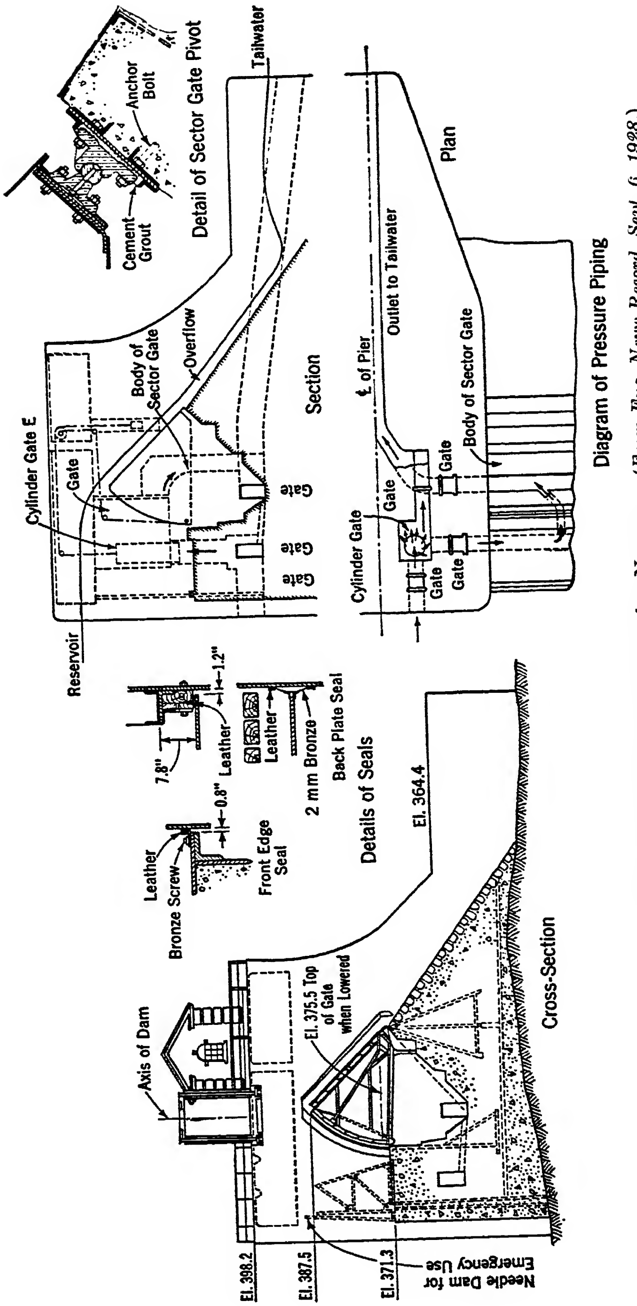


FIG. 10. Cross-section and details of sector gates at Rhananasfos, Norway. (*From Eng. News-Record, Sept. 6, 1928.*)

ports the movable structure against the headwater pressure above the sill at low water. When the water surface rises, the headwater pressure becomes relatively

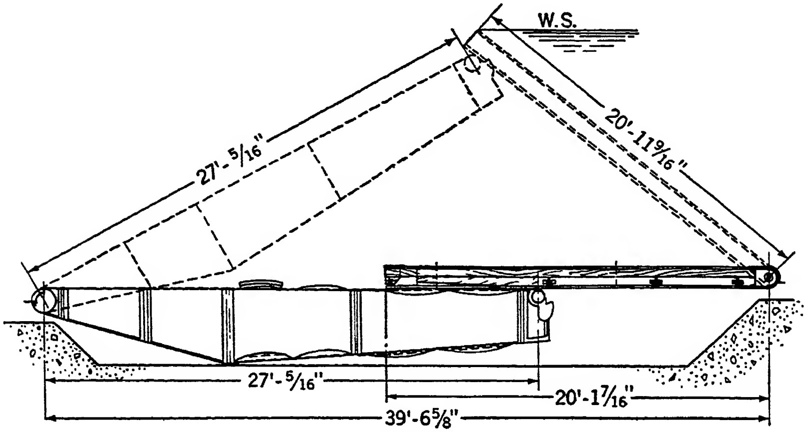


FIG. 11. Bear-trap dam, No. 6, Ohio River. (*U. S. Engineer Office, Pittsburgh, Pa.*)

greater on the upper leaf and the crest is depressed. As the water surface falls to the low water level, the crest returns to that level.

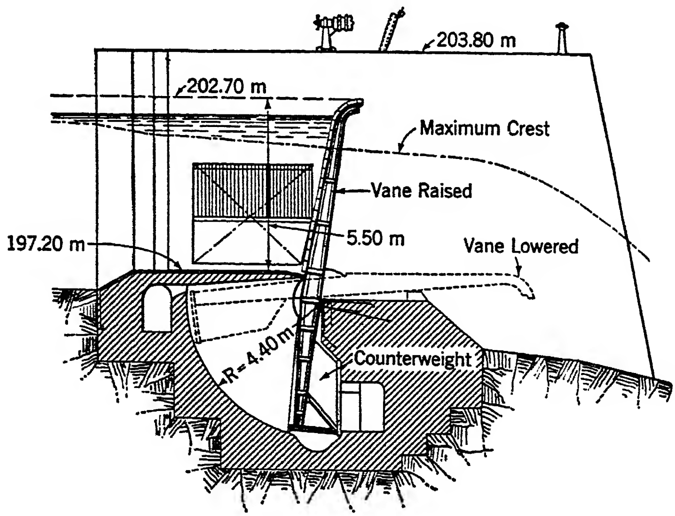


FIG. 12. Automatic spillway gate, Eguzon Dam, France. (*From Eng. News-Record, Vol. 99, p. 460.*)

Fig. 9 shows the type of drum gate used by the U. S. Bureau of Reclamation. It is designed to maintain a controlled crest level and regulated discharge. Control is effected automatically through a device which admits or releases water

rom the chamber under the drum under the influence of the headwater pressure. When lowered, the top surface of the drum matches the curve of the spillway.

Fig. 10 shows a European type of drum gate, commonly termed a sector gate, which is hinged at its downstream side.

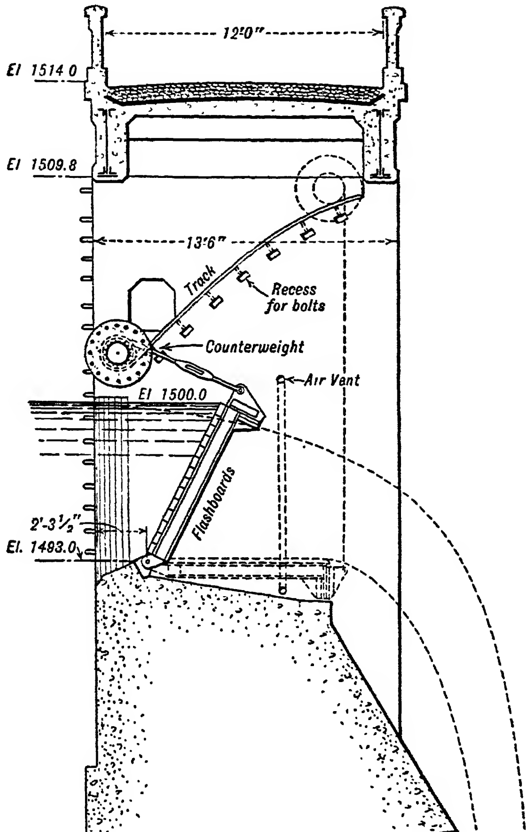


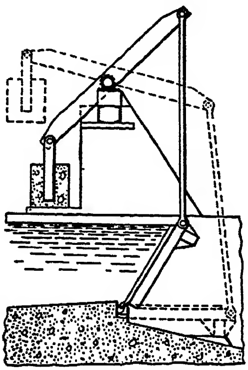
Fig. 13. Tilting gate for Tallulah Falls development. (*Eng. Record, Vol. 69, p. 326, March 21, 1914.*)

(*d*) *Bear-trap dams.* Bear-trap dams were originally developed for sluicing logs through dams used in logging operations on small streams. They have been used for stage regulation on low-head dams, particularly in connection with the movable dams for navigation on the Ohio River.

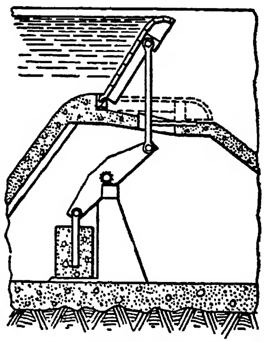
The Ohio River type of bear trap is shown in Fig. 11. As in the drum gate, the surfaces and weights of the moving parts are proportioned so that the headwater pressure supports the damming structure. The trap is raised by admitting headwater to the chamber below the leaves and lowered by draining the chamber to tailwater. Bear-trap dams are ordinarily operated in either the fully

raised or fully lowered positions. They can be raised against the weight of overflowing water by the headwater pressure with usual head difference but are arranged to be raised by buoyancy produced by introducing air under the lower leaf when the head difference is small. Bear-trap dams require a wide base but avoid the use of a deep chamber. They are therefore particularly adapted to applications where the fixed crest level is little above stream bed.

(e) *Tilting gates.* In the tilting gates, the damming surface is hinged at the sill level and is designed to retain the pond at its normal level but to yield under the load of a greater head. Tilting of the gate lowers the effective crest level and releases water. When the reservoir level has fallen sufficiently, the gate returns



This type used for low dams.



This type used for dams of such height that counterweight will not be submerged.

FIG. 14. The Stauwerke tilting gate. Fargo Engineering Co., Jackson, Mich., Agents.

to its raised position. A variety of gates of this type have been installed and many have been patented.

In the example shown in Fig. 12, the force used to oppose the pressure of the reservoir is derived from headwater pressure acting upon an extension of the leaf below the point of rotation.

In the examples shown in Figs. 13 and 14, weights are used to provide the counterforce. The proportioning of the mechanisms controls the discharge characteristics of the gates.

5. Crest Gates. The term "crest gates" is used to designate that class of spillway control in which the damming surface is raised to permit discharge between its lower edge and the fixed crest or sill. Both the water load and the weight of the gates when raised are carried on piers that are located at appropriate intervals along the crest. Crest gates are ordinarily operated by externally applied power and require intelligent control, although they can be, and sometimes are, operated by automatic devices under the influence of the elevation of headwater.

The common types of crest gates are plain sliding gates, roller-bearing gates, fixed-roller gates, and truck gates in which the damming surface rises in a more or less vertical plane, taintor gates in which the damming surface rotates about an

excessive loss of water. They are raised to pass floods. An example of this type is shown in Fig. 19.

(b) *Plain sliding gates.* Plain sliding gates consist of a simple flat damming structure supported between piers. In operation, the friction due to the water

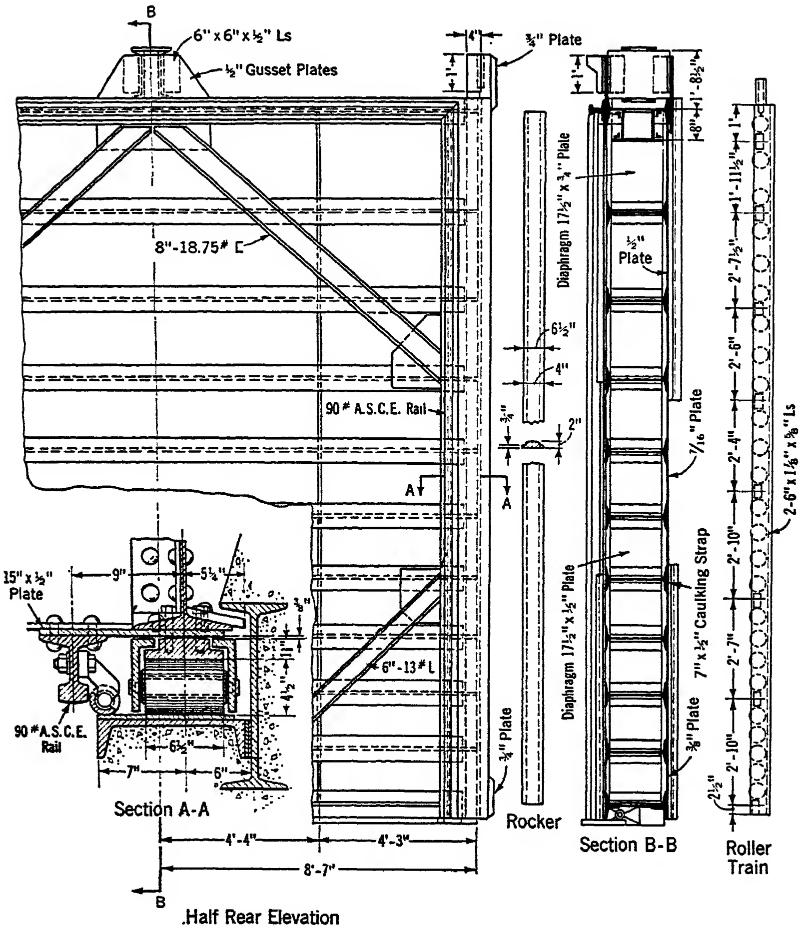


FIG. 16. Stoney gate for Yadkins Falls, N. C., development, Tallassee Power Co.

load on the damming surface acting upon the bearing surfaces at the ends of the gate must be overcome. For this reason, gates of this kind are limited to very small sizes.

(c) *Roller-bearing gates.* In roller-bearing gates, cylindrical rollers are introduced between the bearing surfaces on the gate and on the pier to minimize the frictional resistance to raising or lowering under the water load. The stoney and caterpillar gates make use of this device.

(d) *Stoney gates.* Stoney gates are distinguished by the use of simple straight roller trains between the bearing surfaces, as in Fig. 16. The structure is usually of steel. The water load is carried on a skin plate and transmitted to horizontal beams which are spaced in accordance with the water loading and carry the load to suitable vertical end framing. Vertical tracks on the downstream side of the end beams ride upon the rollers, which in turn roll upon fixed tracks in the piers.

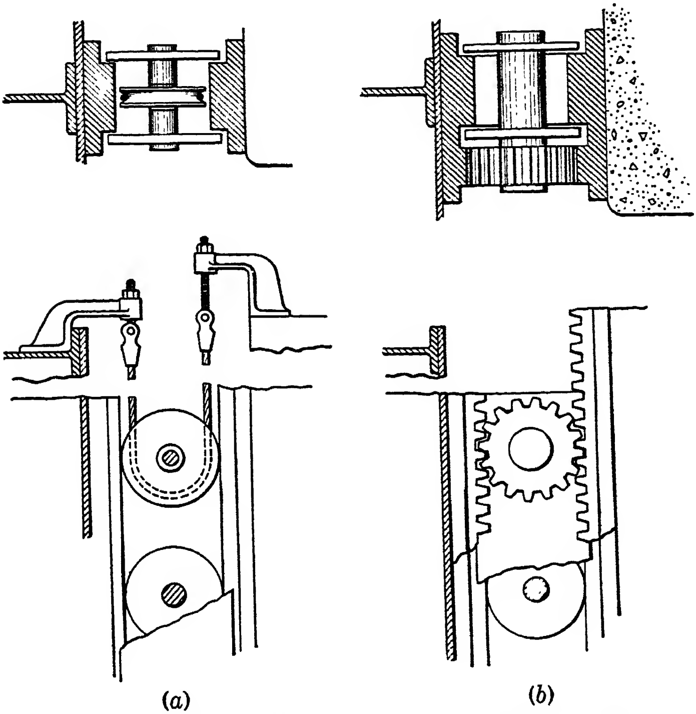


FIG. 16A. Means of regulating travel of stoney gate roller trains.

The rollers are assembled in "trains" to maintain correct spacing and alignment and to support them in position when the water load is relieved. The train is usually formed by positioning the rollers between two steel bars. Holes bored in the bars at suitable intervals receive turned extensions of the rollers or pins passing through the rollers. The roller train is usually positioned laterallv by sliding contact between the side bars and the track on the gate.

The permissible loading for rollers is expressed by the following formula:

$$P = LD \times C$$

when P = load in pounds,
D = diameter of roller in inches,
L = length of contact in inches,
C = constant, dependent upon materials.

For rollers and tracks of carbon steel of about 170 Brinell hardness, a value of 600 for C has been found to give satisfactory service.

Some means is needed to maintain the roller train in proper position relative to the two tracks through the range of travel. The roller train travels only half as far as the gate. A common device for this purpose is a two part wire rope support for the roller train, as shown in Fig. 16A (a). The rope is fixed to a point on the gate, passes around a sheave fixed to the roller train, and is dead-ended on

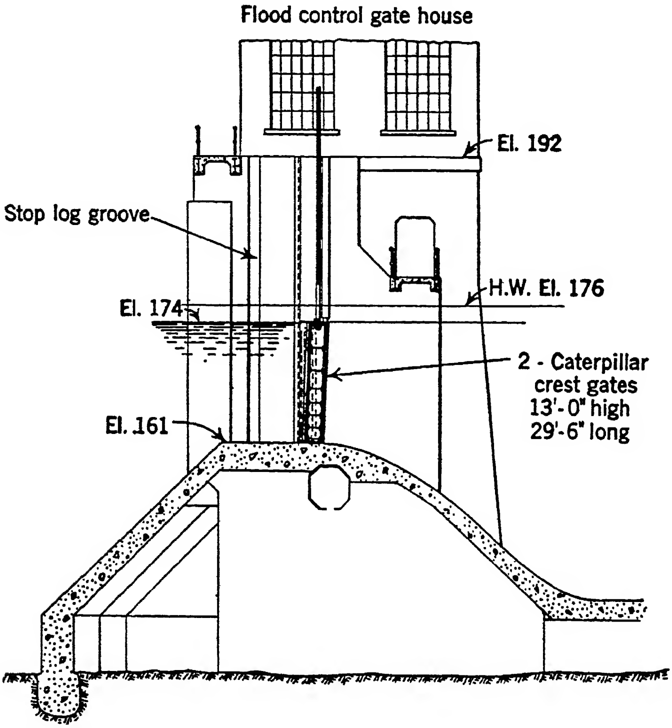


FIG. 17. Caterpillar crest gates, Waterworks Reservoir Project, Oklahoma. (*Philips & Davies, Inc.*)

the pier. A more positive device is provided by mounting on the roller train a gear pinion which engages a gear rack on the gate and one on the pier, as shown in Fig. 16A (b).

Means should be provided for holding the gate within a permissible range of lateral movement. Usually a single roller or sliding surface is provided at each end to contact guide surfaces on the piers. If two rollers or contacts on each end are used, investigation should be made of the effect of the moment that may be produced by diagonally opposite contacts if the gate is lifted unevenly.

(e) *Caterpillar gates.* Caterpillar gates (Fig. 17) are essentially the same as stoney gates except that the roller trains are arranged as continuous chains, as

shown in Fig. 44. The rollers are connected by links, and the assembly travels around a continuous track that is framed into the end of the gate. The Broome type gate makes use of this construction and, in addition, seats along its bottom and sides in a plane that is inclined relative to the plane of the tracks. This arrangement permits positive metallic contact on the sealing surfaces.

(*f*) *Finish and alignment.* Accuracy of finish and alignment of tracks and rollers are of primary importance in roller-bearing gates. The basic idea of this

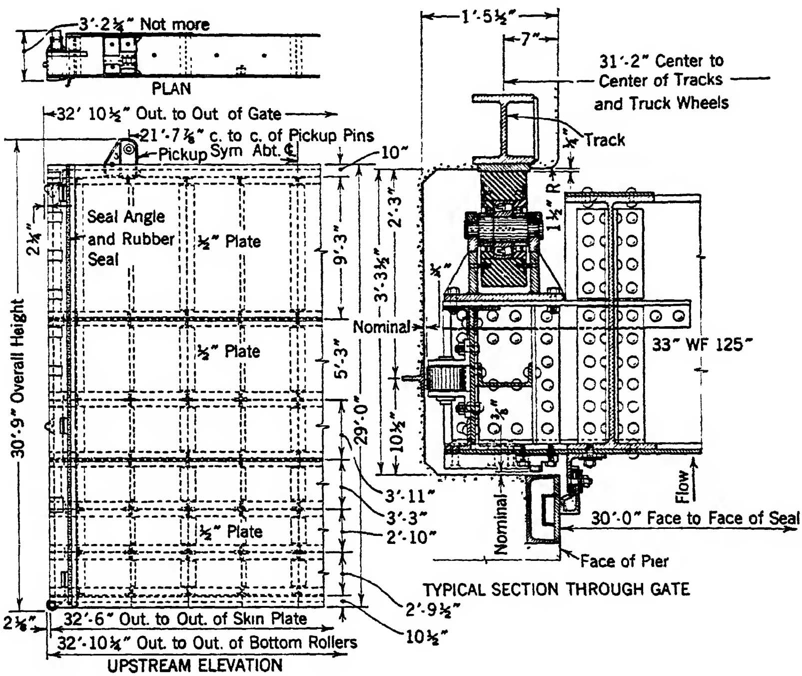


FIG. 18. Fixed roller gate, Mahoning Dam, Mahoning Creek, Pa. (*U. S. Engineer Office, Pittsburgh, Pa.*)

type of gate bearing is predicated upon each roller carrying its proper share of the load. The degree of accuracy necessary for meeting this requirement absolutely is impossible even with the best workmanship. However, in actual practice the flexibility of the gate structure and the compressibility of the rollers and tracks keeps the overloads on rollers within safe limits if reasonable accuracy is maintained in construction and erection.

(*g*) *Fixed roller gates.* In fixed roller gates, the water load is transmitted from the gate structure to the tracks on the piers through rollers or wheels whose axes are fixed relative to the gate. Otherwise this type does not differ essentially from the stoney gate.

The rollers are generally spaced so as to carry more or less equal loads under the maximum load condition. Their capacity may be figured in the manner

previously described for stoney gate rollers except that additional consideration must be given to their bearings and axles. Roller bearings are generally used. The chief objection to plain bearings for this use lies in their high frictional resist-

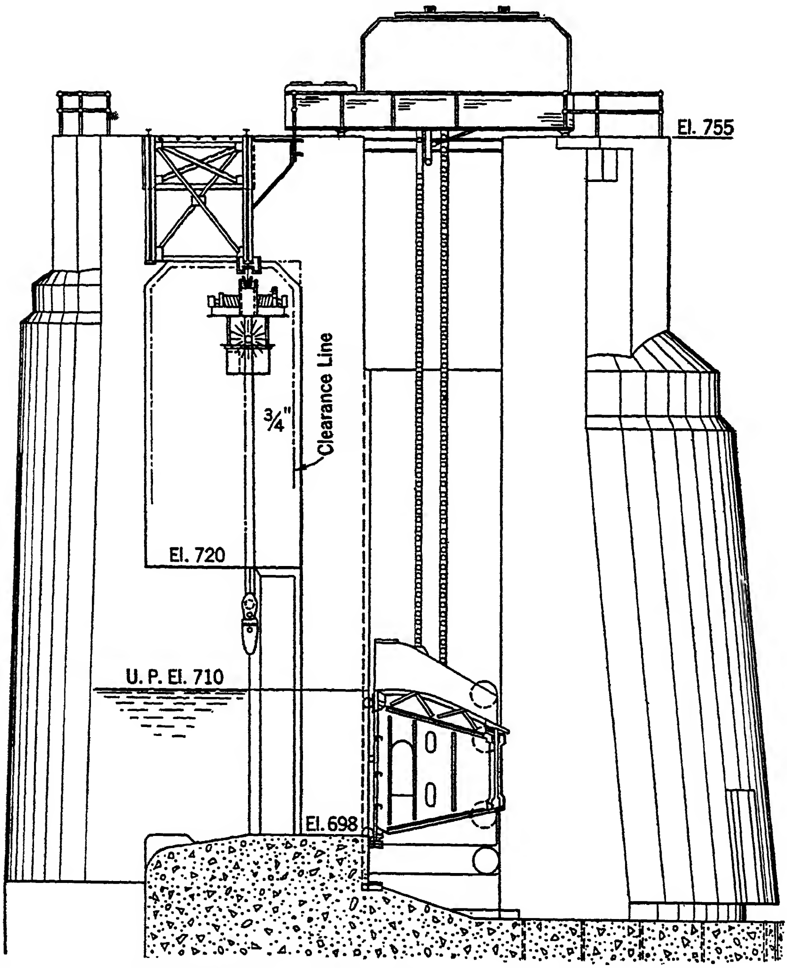


FIG. 19. Emsworth gate hoist Emsworth Dam, Ohio River. (*U. S. Engineer Office, Pittsburgh, Pa.*)

ance when starting under load. Several types of commercially available roller bearings will give satisfactory service in this application if properly arranged and protected.

The mounting of the rollers should be designed so that eccentric loading will be avoided. Eccentric loading may result from the deflection of the gate structure under load or from unavoidable irregularities of the track. Self-aligning roller bearings are usually preferred for this reason.

The roller bearings must be protected from water, air, and abrasive materials. The bearing chambers should be kept packed with grease and are generally sealed by watertight closures which fit snugly about the axles to exclude water and air and to retain the grease.

(*h*) *Truck-mounted gates.* Truck-mounted gates are used where unusually long and relatively shallow gates are needed, particularly in rivers where large

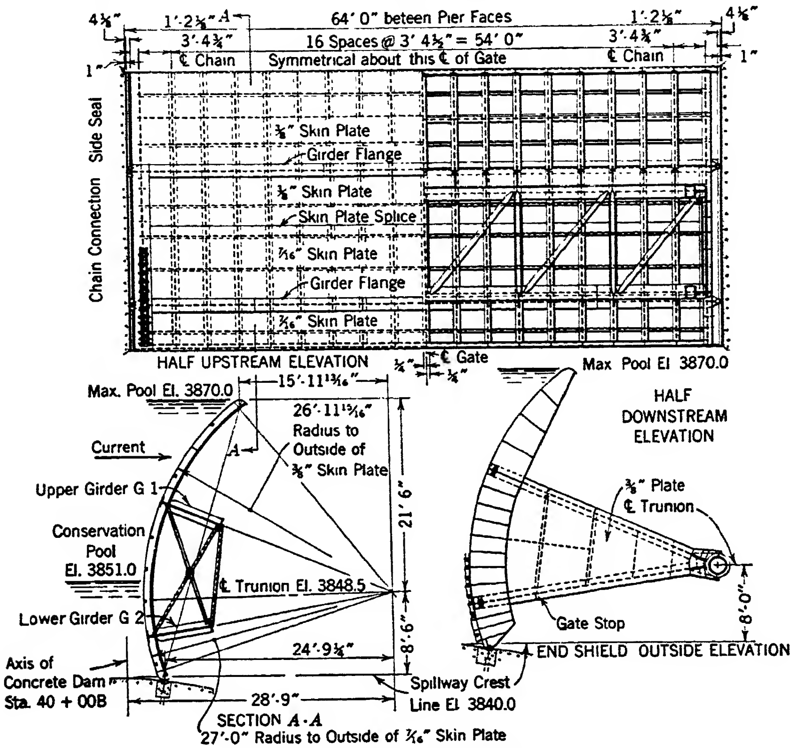


Fig. 20. Taintor gate, John Martin Reservoir, Arkansas River, Colorado. (*U. S. Engineer Office, Caddoa, Colo.*)

drift or heavy ice may be encountered during floods. In such gates, the effects of deflection under the water load and of variations from level in operation are more pronounced than in gates of more usual proportions. The truck mounting provides for the necessary flexibility and is particularly adaptable to a rugged type of construction.

Fig. 19 shows a truck-mounted gate as installed at Emsworth Dam on the Ohio River. The main structure of the gate consists of two longitudinal trusses with a skin plate on the upstream surface and curved overflow plating on the top. The skin plating and trusses are designed to resist the horizontal and vertical bending moments. The horizontal water load is transmitted from the gate to the pier through four two-wheel trucks, two at each end of the gate. The load is

suspended horizontally from the trucks through eyebars so that the gate may swing in a horizontal plane relative to the trucks through a limited range. This mounting allows for expansion and contraction of the gate and its deflection

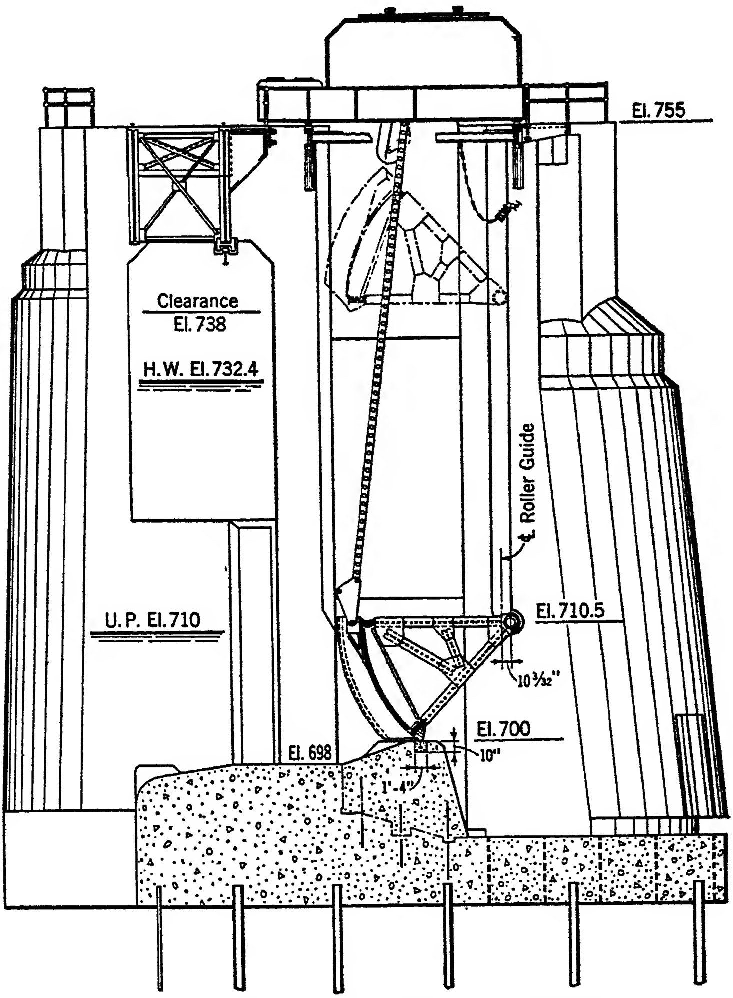


FIG. 23. Sidney gate. (*U. S. Engineer Office, Pittsburgh, Pa.*)

under load as well as minor misalignment of the tracks. It also permits one end of the gate to be raised or lowered as much as 2 ft relative to the other end without affecting the bearing of the wheels upon the tracks.

(*i*) *Taintor gate.* In the taintor gate, the damming surface is a section of a cylinder which is arranged to rotate about a fixed horizontal axis and is generally concentric with it. Its merit lies in its mechanical simplicity. The load is

carried on two bearings, one at each end of the gate on the axis. The bearings are mounted on piers located at appropriate intervals on the crest.

Taintor gates are usually constructed of steel. Small taintor gates have in the past been built of wood, and wood facing has been used on steel framing in the interest of economy and to minimize the collection of ice upon the face.

In the usual design of taintor gates, the water load is transmitted from the facing to horizontal beams that are spaced on the cylindrical surface in accordance with the distribution of the maximum water loading. The beams connect to frames at the ends of the gate which are supported by arms from the bearings.

Taintor gates are usually raised by means of ropes or chains acting simultaneously at both ends. Since the angular travel in the bearings is small when the gate is raised from the closed to opened position, the work done in overcoming the frictional resistance is small.

Fig. 21 shows a typical taintor gate of the usual design.

(*j*) *Sidney gate.* The Sidney gate shown in Fig. 23 retains the essential features of the taintor gate but is adapted to relatively long spans and conditions where the gate must be raised to clear stages much above the normal reservoir level. The main framing is triangular in cross-section, and the main tension member which serves as the downstream chord of the top and bottom trusses is coincident with the axis. In ordinary operation, the damming surface is raised in rotation about the axis, as are taintor gates of the usual design. When it must be raised to clear flood stages, the entire gate is lifted vertically, the bearings leaving their seats and rising along the tracks provided in the piers to guide them. An experimental gate of this type installed at Emsworth Dam on the Ohio River in 1937 has given good service.

(*k*) *Rolling gates.* Rolling gates are generally used for relatively long spans and where severe ice or drift conditions are encountered. Their characteristic feature is a cylindrical beam which spans the opening and rolls up and down on inclined tracks on the piers at the ends, as shown in Fig. 24. A wheel of somewhat greater diameter is fitted at each end of the cylinder and rolls upon the track on the pier. These wheels have coarse pitched teeth which engage depressions or holes in the track surfaces as gears to cause equal travel of both ends. The raising force is usually applied at one end and the force needed to raise the other end is transmitted in torsion through the cylinder. The diameter of the cylinder is ordinarily less than the damming height, and the retaining surface is completed by an apron which, when the gate is in the closed position, extends from the cylinder to the sill and, in some cases, by an extended surface over the cylinder. Relatively large shields cover the recesses at the ends and carry the end seals.

The cylindrical structure is built up of steel plates with longitudinal stiffeners attached to the inside surface. Diaphragm frames are located at intervals, and solid web diaphragms are usually placed at the wheel and chain drum locations. Aprons shaped to give the most favorable water reactions when raising and lowering are constructed of steel plates with suitable bracing and stiffeners. Wheels, tracks, and chain drums are generally made of cast steel.

Rolling gates have been mounted so as to rotate below the normal closed

position and have been fitted with movable flaps on the top of the cylinder to pass ice or drift from the surface of the reservoir.

(*l*) *Operation of crest gates.* The primary function of a device for operating crest gates is generally the development of a large lifting force with a relatively small rate of power input. The force is required to overcome the weight of the

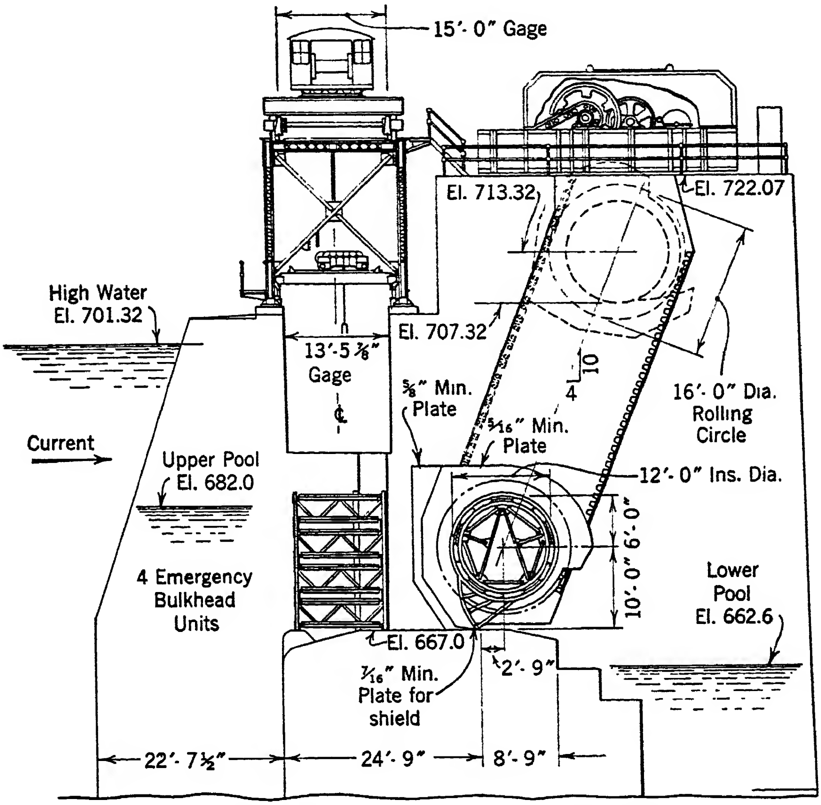


Fig. 24. Rolling gate. (*U. S. Engineer Office, Pittsburgh, Pa.*)

movable parts, the friction of the sliding and rolling contacts under load, the friction of seals, and incidental loads such as the accumulation of ice or sediment on the gate structure or possibly the weight of overflowing water. In the rolling gate, there is also the force of water reaction to be overcome. Ordinarily, the rate of travel is low and therefore the power input is comparatively small, necessitating a high ratio of speed reduction between the initial and final motions.

The rate of travel is usually determined from the maximum rate of increase in outflow that the spillway may be called upon to accommodate. The gate operating devices must be capable of raising the gates clear of the water before the reservoir rises to a dangerous level with the maximum possible rate of inflow.

This consideration, together with the lifting force required, determines the amount of power that must be available for operation.

Small gates are often operated by hand power, and gates of larger size are sometimes arranged so that they can be operated by hand power if necessary.

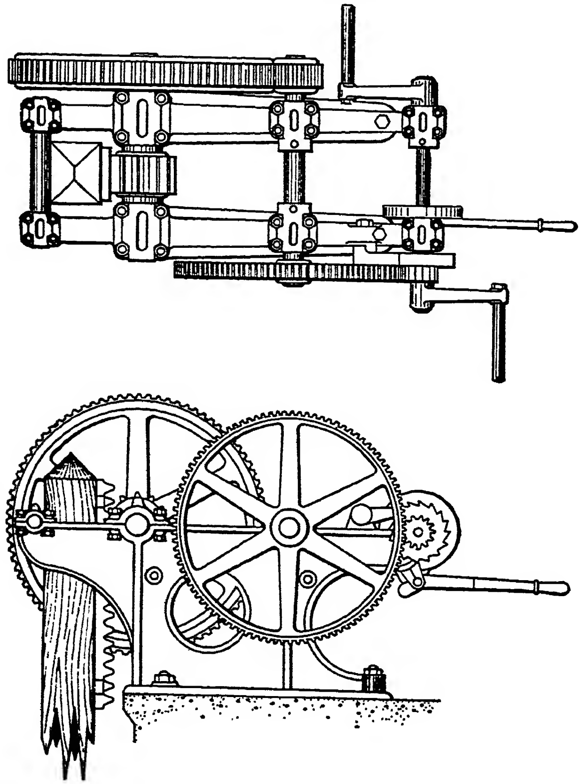


FIG. 25. Small gate hoist. (*From Bull. 137, p. 12, published by S. Morgan Smith Co., York, Pa.*)

For large gates, the possible rate of operation by hand becomes so slow as to eliminate this means from practical consideration.

The usual sources of power for the operation of crest gates are electric energy and internal combustion engines. The former is the most common as the regular source, and the latter is more often used for emergency operation.

Various means are practicable and have been used for converting the power into force and movement of the necessary values. The usual means are mechanical gearing, including screws, spur and worm gear reductions, chains and sprockets, wire rope falls, and hydraulic hoists.

Crest gates are operated either by individual hoists permanently mounted on the gate piers or by movable hoists arranged to travel upon a bridge and to serve

a number of gates. Long gates, such as the truck gates described in Art. 5*h*, the Sidney gate described in Art. 5*j*, and the rolling gate described in Art. 5*k* are operated by individual hoists. Roller-bearing gates, fixed-roller gates, and

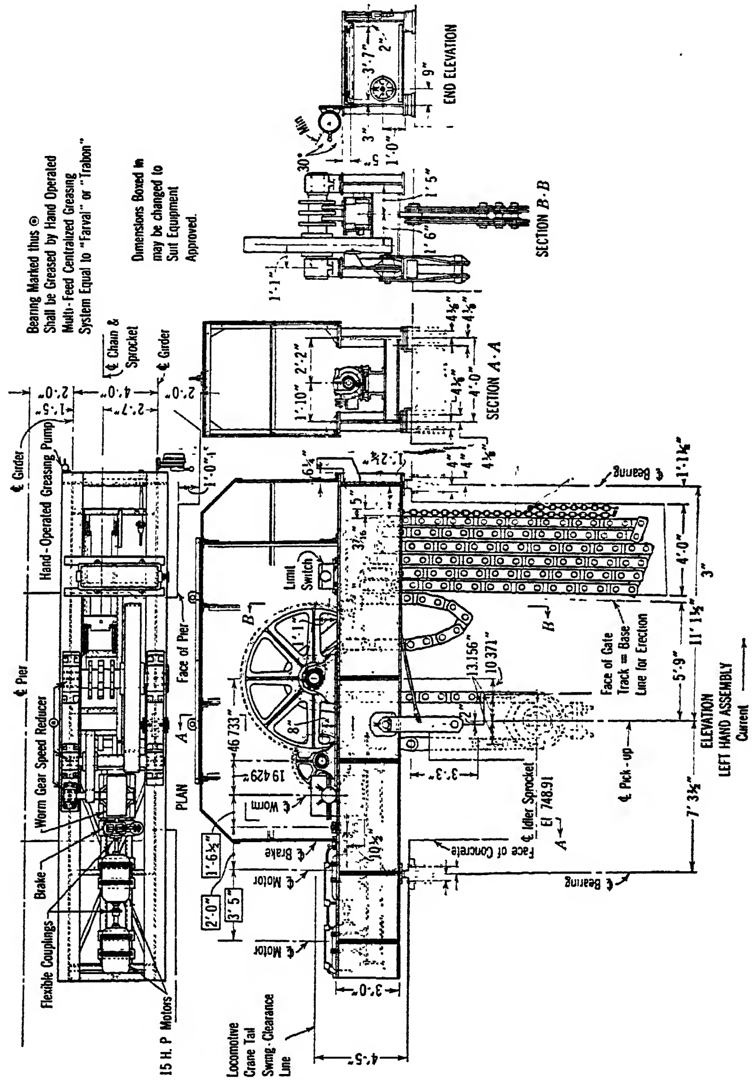


Fig. 26. Gate hoist, Emsworth Dam, Pa. (*U. S. Engineer Office, Pittsburgh, Pa.*)

taintor gates are in some installations operated by individual hoists and in others by movable hoists. Choice between these two arrangements for operation is influenced mainly by the cost of the initial installation and of maintenance and the speed and frequency of operation required.

When individual hoists are used on roller-bearing, fixed-roller, truck, or taintor gates, the lifting force is applied to both ends by separate hoisting devices. These devices must be tied together to insure equal travel. For short spans, the tie is usually mechanical and one motor is frequently used to drive both hoists. For long spans, electrical synchronization of the driving motors is sometimes used.

Fig. 25 shows an operating mechanism for a small gate.

Fig. 26 shows a hoist for a truck-mounted gate at Emsworth Dam.

The form of traveling hoist most commonly used for the operation of roller-bearing and fixed-roller gates is the gantry crane. The points of suspension are placed high enough and the framing of the crane is arranged so that a gate can be raised to clear the tops of the piers and carried across the bridge to other gate bays or to a work bay outside of the range of gate bays. When gates are arranged in a single line along the crest, the position of the hoists may be fixed on the crane. Where gate positions are placed in two lines, as when recesses are provided for regular use and emergency use in each gate space, the hoists must be arranged to travel in an up-and-downstream direction upon the gantry frame. A crane of this type is shown in Fig. 27.

The lift is usually effected by means of wire rope falls. Two sets of falls are used in order to apply the lift as equally as possible to both ends of the gate. Means should be provided for conveniently adjusting the lines to assure a level pick-up.

(*m*) *Pick-up beam.* A pick-up beam is frequently employed to effect the connection between the falls and the gate. The beam is attached to the lower blocks of the falls and is fitted with hooks or retractable pins which engage suitable provisions on the top of the gate. The engaging devices are arranged so that they can be controlled from the crane or the bridge. Fig. 28 shows a pick-up beam designed for such use.

(*n*) *Counterweighted gates.* Counterweights are sometimes used to minimize the force needed to operate roller bearing and taintor gates. An example of counterweighting is shown in Fig. 29. Ordinarily, the weight of the moving parts of the gate is balanced by the counterweight, leaving the friction of the working parts and seals to be overcome by applied power. Such an expedient is useful where gates are to be operated by hand or where limited power is available but, ordinarily, the value of the saving in power is more than offset by the initial cost of the counterweight and its supporting structure.

(*o*) *Automatic gates.* The use of counterweights in conjunction with the effect of buoyancy makes possible automatic operation without power from an outside source. An example of this type of operation applied to a taintor gate is shown in Fig. 29. Fig. 30 shows a stoney gate operated by this means.

(*p*) *Gate seals.* Fig. 31 shows the types of seals most commonly used on crest gates. Direct metallic contacts, such as shown in Fig. 31*a*, require accurate workmanship and may suffer from abrasion where the water carries sand or silt. The poured metal seal shown in Fig. 31*b* presents a convenient method of providing a close-fitting metallic contact. Wood contact surfaces, as shown in Figs. 31*c* and 31*d*, give good service where they are not allowed to dry out.

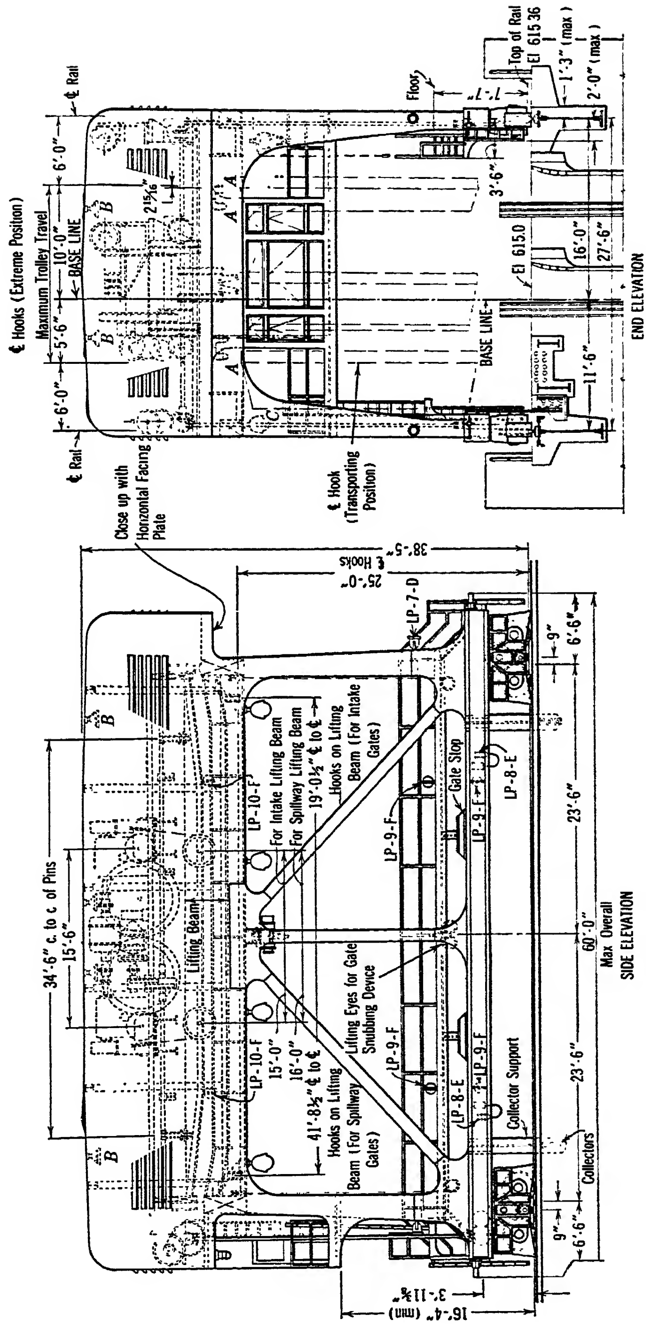


FIG. 27. Gantry crane. (*Tennessee Valley Authority, Knoxville, Tenn.*)

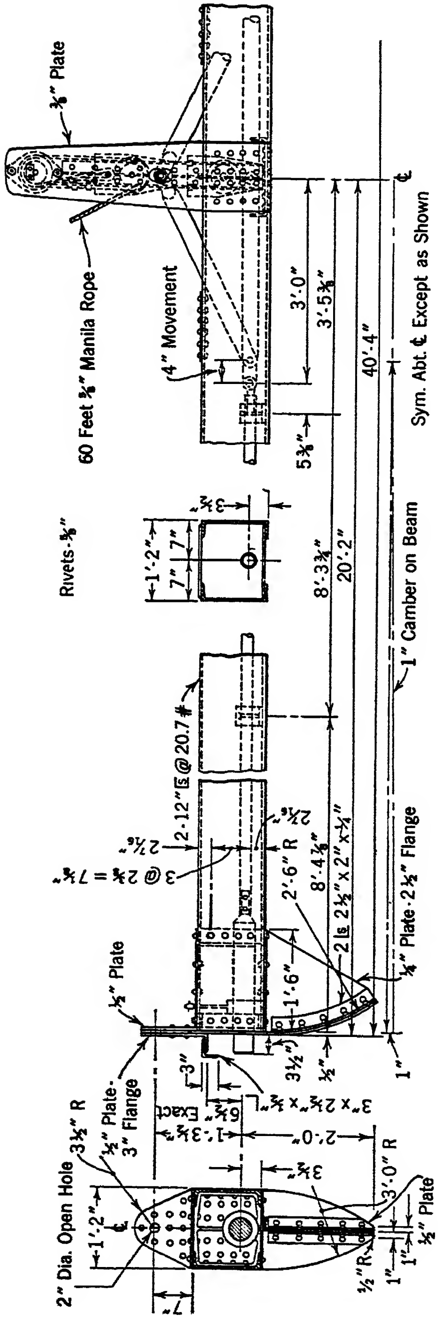


Fig. 28. Pick-up beam. (*U. S. Engineer Office, Pittsburgh, Pa.*)

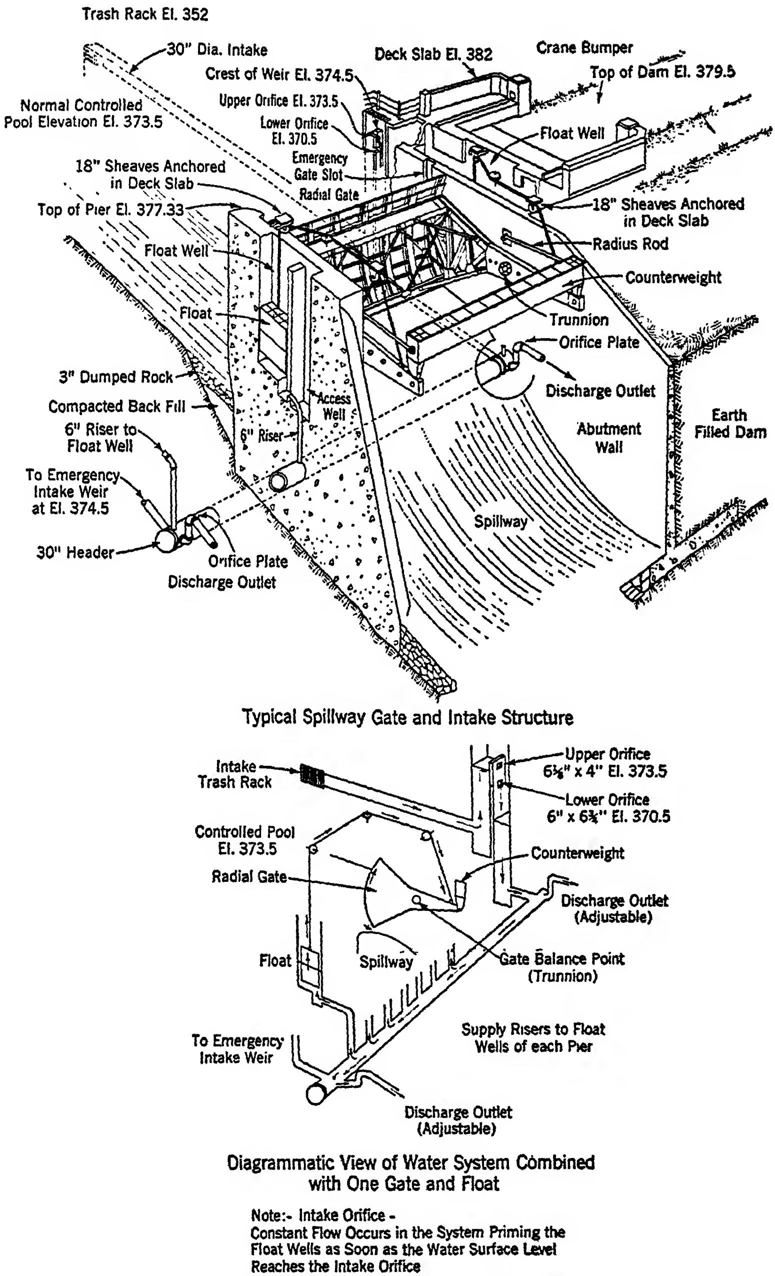


FIG. 29. Automatically operated radial gate. (*U. S. Engineer Office, Portland, Oreg.*)

Flexible rubber sealing devices, as shown in Figs. 31*e*, 31*f*, and 31*g*, are sometimes added to the bottom contacts of crest gates to secure greater watertightness. Side seals are usually of the flexible type. Figs. 31*h*, 31*i*, and 31*j* show typical

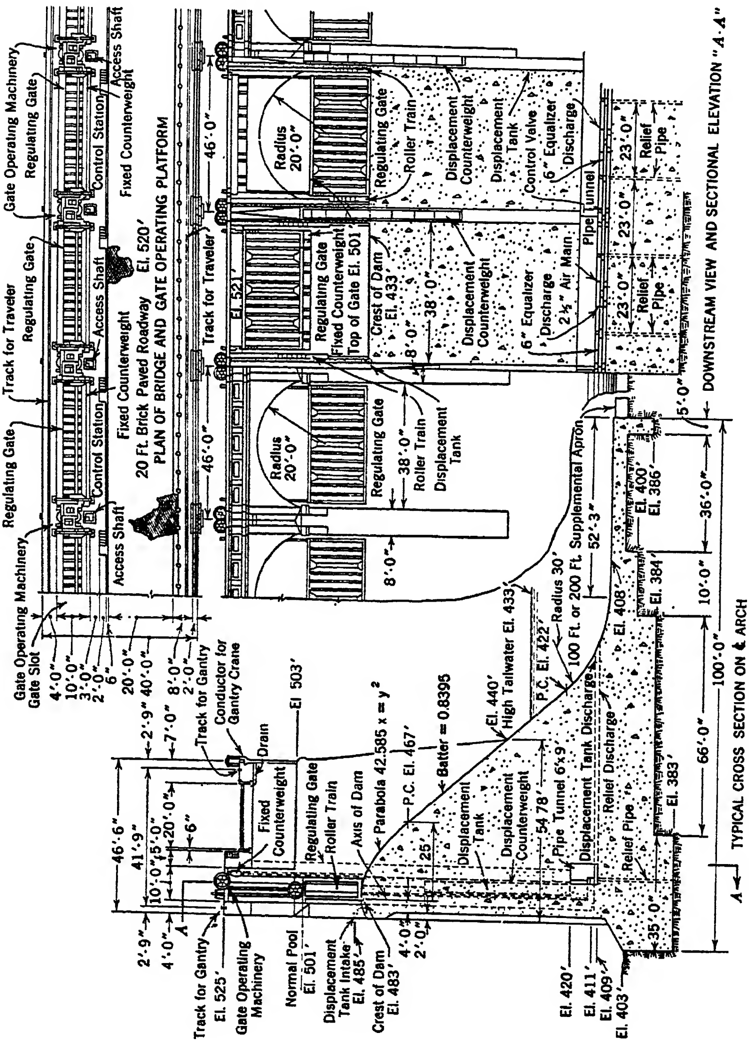


FIG. 30. Arrangement of crest gates at Wilson Dam, Alabama, showing operation by means of displacement counterweights. Details of gate shown on Fig. 15. (*U. S. Engineer Office, Nashville, Tenn.*)

rubber seals. The flexible plates with wood contacts, as shown in Figs. 31*k* and 31*l*, are used where operating conditions are severe. The arrangement shown in Fig. 31*k* is used for rolling gates. The arrangement shown in Fig. 31*l* is used on the Emsworth gates, which are submergible below the sill level.

Steel pipe, wooden needles, rubber hose, and Manila rope are sometimes used as temporary or occasional seals.

(q) *Force required for raising crest gates.* The principal forces that must be overcome by the hoists are the weight of the movable parts, the bearing friction, and the seal friction. Ordinarily, the static or starting friction rather than the

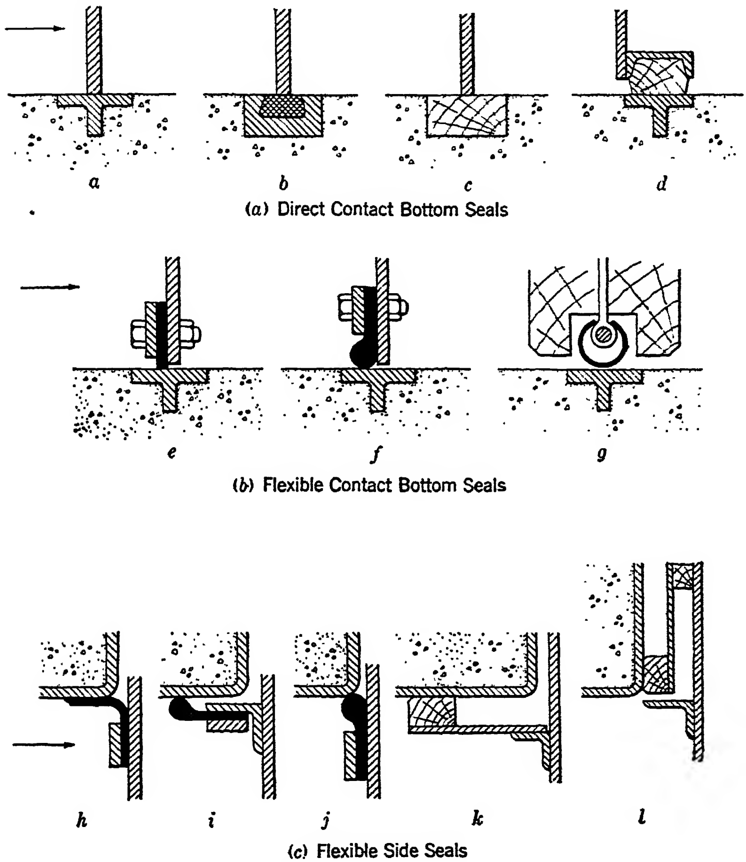


FIG. 31. Typical forms of seals for crest gates.

moving friction determines the operating force. In some cases, additional lifting force is required for accumulation of ice or silt or for the weight of overflowing water.

The bearing friction is generally proportional to the water load on the face of the gate and is dependent upon the type of bearing upon which the water load is supported.

In plain sliding gates, the coefficient of friction depends upon the materials in sliding contact and the condition of their surfaces. When metals of similar structures are used for both the sliding and fixed contacts, the surfaces are likely

to gall when operated in sliding contact under moderately heavy pressures, although certain bronzes can be used in this manner. Ferrous metals subject to rust may develop excessive friction due to the accumulation of rust or pitting where they are exposed to alternate wetting and drying. Recommended allowances for the coefficient of static friction between well-finished clean surfaces of commonly used materials are given in Table 2. Dry surfaces are used in the table, since the lubricating effect of water is problematical where two surfaces are squeezed together tightly.

TABLE 2

RECOMMENDED ALLOWANCES FOR THE COEFFICIENTS OF STATIC FRICTION FOR SMOOTH-FINISHED DRY SURFACES

Steel on steel	0.6	Wood on metal	1.00
Steel on cast iron	0.6	Wood on wood	1.10
Steel on bronze	0.45	Rubber on metal	1.10
Bronze on bronze	0.45		

Stoney gates, caterpillar gates, and fixed-roller gates all bear on some form of roller bearing, and the primary resistance to their movement is rolling friction. The coefficient of rolling friction is low and would not exceed 0.005 with reasonably good workmanship. However, considerable friction may be introduced by the parts used to hold or contain the rollers. In stoney gates, friction is produced in the bearings of the roller axles in the side bars or rollers. Likewise, the links and pins used in the caterpillar roller train introduce a friction load. In fixed roller gates, friction occurs in the contact of the closure seals of the rollers and in the lubrication in the bearing spaces. The amount of friction load that may be accumulated from these sources is dependent largely on the condition of the parts and is quite variable. For gates of this type, it is usual to allow for a load amounting to 5 per cent of the water load on the face of the gate for the bearing friction.

In the taintor gate, the operating force has the advantage of leverage over the frictional resistance at the bearing. The bearings are usually of the plain cylindrical type arranged for grease lubrication. When the gate stands for a time under load, the lubricant is squeezed out from between the contacting surfaces and the coefficient of friction approaches the value for unlubricated surfaces. Referring to Fig. 32, the operating force required to overcome the bearing friction may be expressed as follows:

$$F = \frac{KPr}{R}$$

when P = total water pressure on gate,
K = coefficient of friction,
R = radius at which F is applied,
r = radius of bearing.

Seal friction is dependent upon the pressure exerted upon the seal contact and the coefficient of friction between the sealing surfaces. In flexible seals, the force

exerted at the seal contact is usually derived from the headwater pressure, although mechanical springs have been used to increase the contact pressure. In rubber seals, closure is usually effected close to the headwater side of the contact area, and the headwater pressure should be considered as acting over the entire seal area. The less flexible wooden seal surfaces may have their line of closure at any place in the contact area and therefore it is safer to assume, when estimating the seal friction, that the headwater pressure acts over the entire contact area. The coefficient of friction between rubber and smooth metal surfaces is low as long as the contact is lubricated by a thin film of water. In gate seals, water is not retained between the contact surfaces and the coefficient of friction is rela-

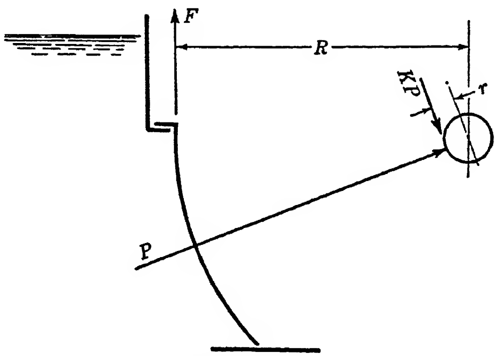


FIG. 32. Forces affecting operation of taintor gate.

tively high. Coefficients of friction for materials commonly used for seals are included in Table 2.

(*r*) *Sizes of crest gates.* The selection of the type of gate for any crest gate installation is influenced to a great extent by the dimensions of the opening that is to be controlled. While no definite lines can be drawn to fix the suitability of certain types of gates within certain size ranges, the dimensions of existing installations indicate the types that designers have found to be most suitable under average conditions. In the range under 60 ft in length and 30 ft in height, taintor gates predominate. Stoney gates also fall within the same range. Fixed-roller gates have been used mostly for heights of 30 ft and greater. On the Tennessee River Dams of the Tennessee Valley Authority the gate openings are 40.5 ft wide and the gates are 40.4 ft high. These gates are divided horizontally into two sections of approximately equal height which are handled separately. The fixed-roller construction used for these gates is particularly suited for this plan of operation. Similar gates on the Bonneville Dam, Columbia River, are 50 ft long and 50 ft high. Rolling gate installations range between 60 and 150 ft in length, with heights up to 30 ft. Truck type gates have been constructed within the range of 60 to 100 ft in length and 16 ft in height. These dimensions do not represent absolute limits for any type of gate, and examples may be found outside of the ranges stated.

(s) *Weights of crest gates.* The available reported weights of various types of crest gates are characterized by the lack of complete information which would enable the published weights to be intelligently applied. The weights of gates involve a consideration of the following: (1) the general type of gate (taintor, stoney, rolling, etc.), (2) the size of the gate, (3) the design loading and the design allowable unit stress, (4) a clear understanding of what the given weight represents (moving parts, embedded or track metal, and hoisting equipment), and (5) special design considerations such as radical departures from conventional types and additions of metal for corrosion or erosion protection.

The formulae presented herewith have been developed by Mr. M. V. Harrington, Engineer, U. S. Engineer Office, Pittsburgh, Pa., and, except for the rolling gates, have been derived, using empirical constants and based on the weight varying directly with the water load on the gate and directly with the square of the span length. The constant in the term (L^2 + constant) provides for an increased proportional weight of the end framing, mounting, etc., as the span decreases.

The weights of the rolling gates appear to follow a straight line variation dependent only on the height and length of the gates. The individual considerations determining the shield proportions and rolling circle sizes cannot be considered in a simple formula.

There is an acute lack of published data on weights of embedded metal for various types of crest gates. The formulae given are approximate only, and the user should give consideration to the widely varying unit prices of castings, corrosion-resistant metal, and structural steel.

L = clear span length in feet,
H = height of gate in feet,
h = design head to the bottom of the gate as measured in feet,
Fs = unit design stress of material in psi,
W = gate weight in pounds,
w = embedded metal weight in pounds.

The gate weight formulae are intended for use only for ratios of h/H of not more than 1.5.

1. Radial or taintor type of gates:

   Moving parts $W = \dfrac{935}{Fs} Hh(L^2 + 700)$

   Embedded metal $w = (L + 2H)125 +$ (225 lb per ft of gate lift)

2. Stoney, fixed roller, etc., type of gates:

   Moving parts $W = \dfrac{750}{Fs} Hh(L^2 + 870)$

   Embedded metal $w = (L + 2H)\,190 +$ (450 lb per ft of gate lift)

3. Truck-mounted gates:

   Nonoverflow, moving parts $W = \dfrac{850}{Fs} Hh(L^2 + 3000)$

   Overflow or submergible, moving parts $W = \dfrac{1970}{Fs} Hh(L^2 + 350)$

Nonoverflow, embedded metal $w = (L + 2H)170 + (350$ lb per ft of lift)
Overflow or submergible, embedded metal $w = (L + 2H)185 + (350$ lb per ft of lift)

4. Rolling gates:

$$\text{Moving parts } W = \frac{2800}{Fs(\text{in kips})} LH$$

Embedded metal (including rack) $w = (L + 2H)250 + (1300$ lb per ft of lift)

(*t*) *Stop logs.* Stop logs are vertical layers of loose timbers spanning an opening between piers or abutments and supported at each end in grooves. A typical stop log layout is shown in Fig. 33. Stop logs, which are removed one by

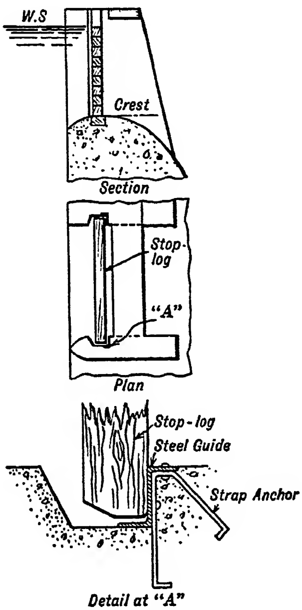


FIG. 33. Typical stop-log layout.

one as the need for increased discharge occurs, are the simplest form of crest gates. The chief objection to their use is the difficulty of installing and removing them.

Fig. 34 shows a winch designed for their removal during periods when water is passing over them. Stop logs are installed by pushing them down into positions one by one with poles, where they are held in place by water pressure.

Stop logs are chiefly used for the outlets of small log and trash chutes and are usually removed by hand. They are extensively used for emergency purposes, to unwater all kinds of water-control apparatus.

The facility with which stop logs can be installed depends greatly upon the smoothness of the guides. Therefore wooden or steel guides are used for long spans and deep sluices, unless special provision is made for placing very smooth

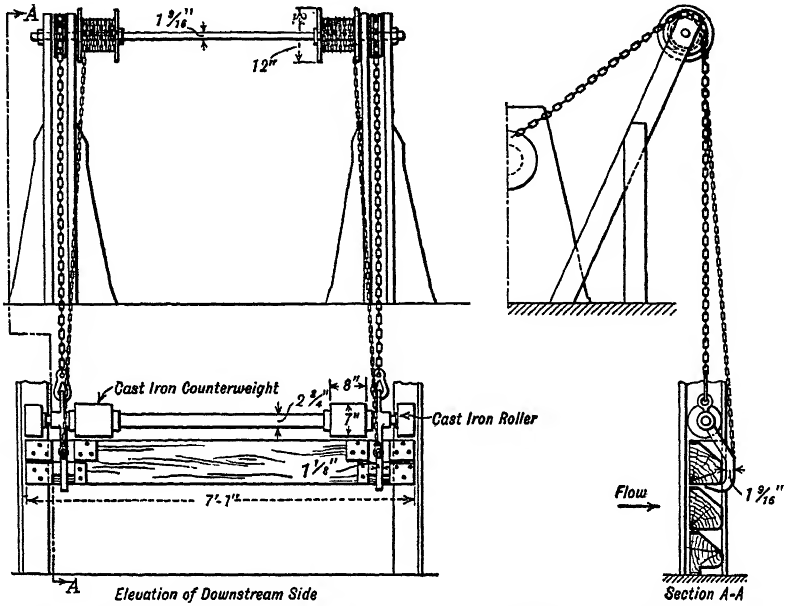


FIG. 34. Weighted hooks for grappling stop-log. (*L. Ross in Eng. Record, Vol. 74, p. 114.*)

concrete for the logs to rest on. While leakage below low-water surface will preserve wooden guides, they will rot at and above water. Therefore steel guides similar to that illustrated in Fig. 33 are recommended.

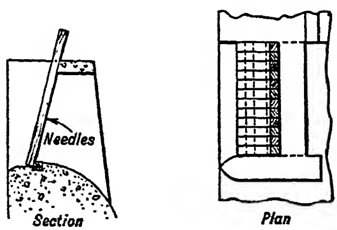


FIG. 35. Typical needle layout.

(*u*) *Needles.* Needles consist of a row of slightly inclined timbers supported at the top by a bridge or beam and at the bottom by a sill on the crest, as shown in Fig. 35. The needles are placed one by one, by extending them horizontally over the pond and allowing them to tip into the current until the lower end swings down in contact with the concrete near the sill. They are then drawn slightly upward until the lower end rests on the sill and are rolled sidewise into place

against those needles already installed. They are removed by lifting each from its seat and hauling it out bodily. Each should be provided with a hole near the top through which to pass an anchor rope. The purpose of this rope is to hold the needle in case it gets away from the operators when being handled. Needles are invariably installed and removed by hand so should not be too large. Timbers 6 by 6 in. and 20 ft long have been used.

This type of control is not frequently used.

6. Siphon Spillways. The siphon spillway is a device for discharging water over a dam. It utilizes the available head of the dam to produce a higher velocity of flow than would be attained at an overflow weir, thus increasing the dis-

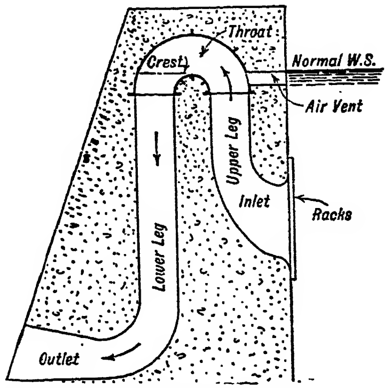


Fig. 36. Simple siphon spillway.

charge for the same elevation of water surface in the reservoir. Thus, for a given length available for a spillway, the use of a siphon spillway will result in a smaller rise of water surface for the same discharge than would be possible for an overflow weir.

Fig. 36 shows the normal headwater surface at the level of the crest of the siphon spillway. When the water rises, it spills over the crest; and, when the flow is such that the discharge strikes the downstream side of the lower leg, the air thus confined in the throat is quickly entrained and ejected, and the siphon primes. The suction thus produced increases the velocity to that corresponding to an effective head equal to the difference in elevation between headwater surface and the center line of the outlet, less the head expended in friction within the siphon.

If the full discharge of all the siphons at the dam is greater than the total flow into the pond, the headwater surface will fall. When the upper parts of the air vents are exposed, the air drawn in by the suction reduces the efficiency of the siphons until the discharge is automatically diminished to that required for stationary water surface in the pond. If, however, too much air is drawn in, the siphon action will be broken and the headwater surface will then rise again, and the operation of priming will be repeated. If properly proportioned, the siphons will prime within a few seconds after the water has risen to the required elevation.

The siphon depicted in Fig. 37 was shown to be capable of priming within 2 sec after the water had risen high enough to effect a seal in the lower leg (1).[5]

The upper leg is made of sufficient length to bring the inlet well below water surface, in order to prevent the entrance of ice and drift. The inlet is made two or three times the area of the throat, is well rounded, and is usually protected by rack bars rather wide apart. The lower leg should be as long as is practicable, up to the siphonic limit, to take advantage of all the head available. The outlet

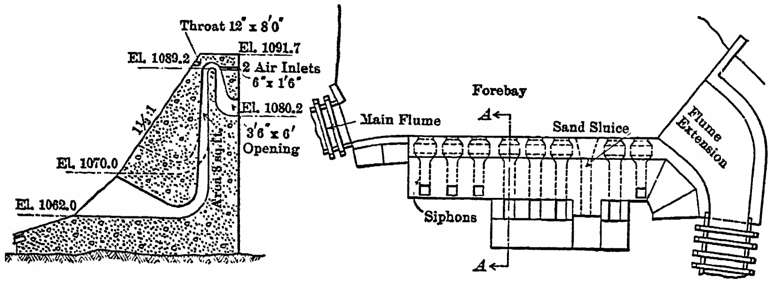


Fig. 37. Siphon spillway, Ocoee River Development, Tennessee. (*Eng. Record, Vol. 72, p. 567.*)

may be submerged or may be opened to the air, except for special cases where submergence is necessary.

Wheaton recommends that the outlet of a siphon with a water seal should be submerged as little as possible, or compression resulting in the tube will cause an increase of both the time and head necessary for priming (2).

The throat is frequently protected by a lining to prevent erosion if the velocity is high. Provision should be made to bypass ice cakes and large pieces of debris which otherwise might clog the siphons.

Siphons with a throat height as great as 8 ft have been used. Figs. 36 and 37 indicate the usual type of siphons, but many other forms have been adopted, as indicated in Fig. 38.

Siphons of the type shown in Fig. 37 usually prime when the water has risen a distance equal to about one-third the height of the throat. For very large throats, the resulting rise may be too great. Therefore some means must be used to obtain a quicker priming. This has been accomplished by providing a small auxiliary priming siphon, having a small height of throat. Such a siphon will prime with a small rise of headwater surface. It is connected to the large siphons by a pipe which draws the air out of the large siphons and in turn causes them to prime with a small rise of water surface. In such cases, the outlet of the large siphons must be sealed with water. However, priming siphons are not now considered necessary. Other methods of providing quick priming action are described in the references indicated hereinafter. These consist, in general, of providing a method of deflecting the spilling water across the lower leg to form a sealing curtain.

[5] Numbers in parentheses refer to Bibliography, Art. 9.

Fig. 37 shows the siphon spillway of the Tennessee Power Co. on the Ocoee River.

The discharge of a siphon may be computed from the ordinary equation for flow through short tubes.

$$Q = CA\sqrt{2gh}$$

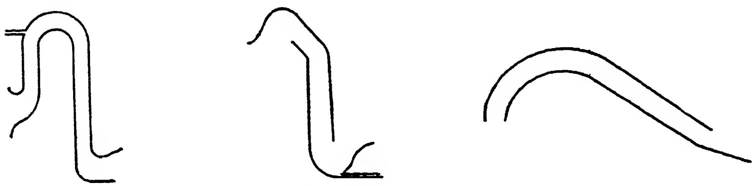
where h = the gross head on the siphons in feet from headwater surface to center line of outlet or tailwater surface, if submerged,

A = the area of the throat, in square feet,

g = the acceleration of gravity = 32.2,

Q = the discharge, in cubic feet per second, and

C = a coefficient depending on the characteristics of the siphon.



(a) (b) (c)

FIG. 38. Diagrammatic sketch of typical siphons.

The throat of the siphon must be so designed that the absolute pressure at the summit is materially greater than zero. To obtain the pressure at the summit, deductions must be made from the atmospheric pressure at entrance for entrance loss, friction between entrance and throat, velocity head at summit (or bottom of throat), difference in elevation, and vapor pressure, etc. Generally, the resulting residue of absolute pressure at any point in the throat should not be less than 10 ft.

The long leg of a high head siphon should be so designed by tapering or a nozzle at the outlet that it will flow full.

Tests by W. P. Creager on the Ocoee siphons indicated a value for C of about 0.65, and this figure has been fairly well substantiated by other tests on siphons of the same type.

Other types of siphons have coefficient C ranging from 0.25 to 0.98 (3). Methods of calculating the coefficient C are identical with the theory of flow of water in pipes and are described in references 3, 4, and 5. Wheaton has shown that the action of siphon spillways can be determined accurately by model tests, provided that models to several scales are constructed and compared above some critical value of Reynolds' number. The break in the curve of coefficient of discharge plotted against operating head seems to occur for models of the same siphon to several scales at a constant value of Reynolds' number. There is probably also a limit to the scale reduction of the model for reliable results (2).

7. Ice Troubles at Crest Gates. Ice must be prevented from forming on crest gates in order to insure their being in operating condition when needed. If neglected in cold climates, ice will form in great quantities below leaky gates, and

the entire upstream face of exposed steel gates will be coated with ice several feet thick. The ice which forms on the face of the gates may be too heavy for the hoist to lift and, moreover, the ice will adhere to the piers and sills.

The operation of removing large quantities of ice from the gates is both slow and expensive. In most cases, quick opening of the crest gates is very necessary to prevent flood waters from rising too high and causing damage. Winter floods are frequent in most of the northern states, and even the spring floods may occur before the ice at the gates has melted away. Aside from the great danger of possible unsuccessful operation of frozen gates when badly needed to pass sudden

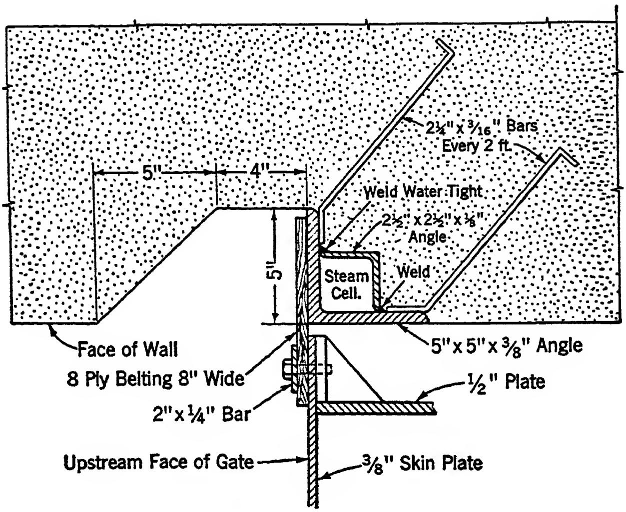


Fig. 39. Arrangement for steaming Taintor gate seals at Chippewa Falls Hydro Plant.

floods, it will be found economical to make provisions to prevent the ice from forming if the gates must be operated during the winter season.

Reference should be made to the November 4, 1924, issue of *Power*, which contains a synopsis of an excellent paper read by Mr. J. S. Bowman before the American Society of Civil Engineers on ice troubles at spillways. This paper has been freely drawn on in this section.

Three methods are used successfully to prevent freezing of crest gates. These are (1) heating by steam, (2) heating by electricity, and (3) the provision of air jets to circulate the headwater.

For the first method, the downstream face of the gate is housed in, usually with two layers of 1-in. sheeting with building paper between, and steam coils are located close to the skin plate of the gate and also embedded in the concrete near the gate seals. Steam must be supplied continuously during cold weather because, if the pipes are allowed to cool, condensation may freeze them solid when steam is first turned on. The 25- by 15-ft taintor gates of the Consumers' Power Co. in northern Michigan required 15 tons of coal each for continuous winter operation. The boiler capacity was 15 hp per gate. Fig. 39 shows the

detail of a heating unit in a recess in the concrete near the gate seals, used by Northern States Power Co.

In some cases, space heaters have been substituted for the steam pipes inside the housing with equal success. Four 500-w Westinghouse space heaters are placed at the bottom of each of the 16- by 12-ft taintor gates of the Jim Falls Dam in Wisconsin, at a total cost of $170 per gate. The heaters require about 5000 kw-hr per gate per yr.

The W. G. Fargo Engineering Co. has made a practice of heating only about every fourth gate of a series. When the heated gates are opened during the winter, the warm water flowing from the bottom of the pond past the other gates will, within 2 to 4 hr, melt the ice on them sufficiently to permit their being opened.

In the third method of preventing ice formation, compressed air, released from the pipes near the bottom of the gate, creates a circulation which draws the warmer water from the bottom of the pond to the surface. This method will not only prevent the formation of ice on the face of the gate and the piers but will keep an area of open water above the gate. It was first used in this country at the Keokuk development of the Mississippi River Power Co.

The following description of the compressed-air installation at Twin Branch was furnished by Mr. J. S. Bowman, Hydraulic Engineer, Fargo Engineering Co., Jackson, Mich.

> In 1923, an air system was installed at the Twin Branch Dam of the Indiana and Michigan Electric Co. from designs of the Fargo Engineering Co. The spillway gates consist of seven taintor gates, 25 ft long and 10 ft 6 in. high. This system was installed not so much to prevent pressure against the gates as to prevent the formation of ice on the upstream face and at the ends and sill.[6]
>
> An old 6- by 6-in. air compressor, used for dusting the generators, was speeded down to 120 rpm and the air passed through a combined cooler and receiver and a reducing valve to a 1½-in. main on the runway over the gates. At each end of a gate is a ⅜-in. needle valve and union for attaching the ⅜-in. aeration pipes which discharge the air into the corner formed by the sill and the pier. The discharge of the air at this particular point was found to be very important. The aeration pipes are in place only during the winter. Air is supplied by the compressor at a pressure of 15 or 20 lb and reduced to a pressure of 5 lb in the main.
>
> Approximately 1.0 cu ft of free air per min was used at each of the 14 outlets during the past winter. The temperatures during the month of January were very low for this locality over a protracted period. On 9 days the minimum temperature ranged from 0° F to −16° F, and on 4 days the daily mean was below zero. For the entire month the average minimum temperature was 10° F and the average maximum temperature was 29° F with a mean of 19° F. Ice formed on the pond to a thickness of 16 in. and, at the Elkhart plant, 11 miles upstream, where all conditions are similar, ice formed on the face of the gates to a thickness of 12 in. However, the air system was kept in operation at Twin Branch without interruption, and with practically no attention the gates were kept entirely free from ice. The current of water came upward at the junction of the gate and the pier, then circulated on the surface toward the

[6] This statement has reference to the compressed-air installation at Keokuk, which was designed to prevent ice thrust against the gates.

center of the gate. In each corner an area of about 2 sq ft remained entirely open, and along the face of the gate the surface was open for a width of 2 to 12 in.

The power required to operate this system was not over 2 kw for the seven gates. At a cost of $0.010 per kw-hr this would cost approximately $0.07 per gate per day with very slight attendance by the operating force.

The ice must be kept clear of the face of the gate, not only to permit operation but also to prevent thrust from the ice sheet against the gates. This thrust has been known to damage the gates to a considerable extent. Care should be taken not to thaw the ice away from the projecting nose of the piers, which should take the ice thrust.

Flashboards require constant attention throughout the winter. Unless the ice sheet is kept cut back from the upstream face, the thrust will cause failure. If the flashboards leak or if water is allowed to trickle over them, large accumulations on the downstream face may prevent temporary flashboards from bending over as desired or permanent flashboards from being removed. The author believes that the compressed-air installation previously described has not been used for flashboard; but it should prove equally successful for this type of crest control.

8. Controlling Devices for Reservoir Outlets. Controlling devices for reservoir outlets may be divided into two classes by the character of service required of them. The first class serves those outlets which are operated occasionally or periodically and which are ordinarily operated with the opening fully closed or fully open. The second class serves to maintain a closely regulated outflow through a restricted opening. With low heads, all types of sluice gates are used for both classes of service. For higher heads, close regulation is effected through valves designed especially for that purpose.

(*a*) *Sluices.* The performance of sluice gates is influenced by the design of the sluice as well as the design of the gate. An abrupt change in the relation of the velocity and static heads takes place where the sluice is restricted by the gate. This causes disturbances in the sluice and frequently produces negative pressures and cavitation in the sluice below the gate. Improperly shaped entrances may cause disturbances which persist through the sluice, and irregularities in the surface of the sluice or gate frame may produce harmful effects. These effects may be minimized by proper design of the water passages.

The entrance to the sluice should be shaped as nearly as possible like that of the standard orifice, and the area of the sluice should be reduced gradually between the entrance and the control. It is now generally conceded that the sluice, from the control to the outlet, should be the same section as the control. Sluices have been constructed with increasing section below the control for the purpose of regaining some of the velocity head and increasing the discharge. However, a large percentage of increase in discharge can be obtained by this means only for very low heads, and the practice is objectionable on the grounds that it increases the vacuum and hence increases the erosion.

A change of section will invariably be necessary at the control. Consequently, if the control is not placed at the extreme lower end of the sluice, the concrete below the control must be well protected by a cast iron or steel lining. The lin-

ing must be securely anchored to the concrete, and the concrete behind the lining must be drained to prevent rupture due to the pressure of seepage from headwater. Drainage may be effected by holes through the lining or by porous drains placed in the concrete behind the lining.

If the tailwater below the dam has considerable depth, the maximum capacity of the sluice is obtained when it is placed just below the tailwater surface. However, in such cases, the sluice control is not accessible for repairs. If the sluice is placed above tailwater, some head is sacrificed and leakage water will freeze in cold climates. If the tailwater fluctuates sufficiently, the sluice may be placed below the water surface corresponding to the river discharge that occurs when the

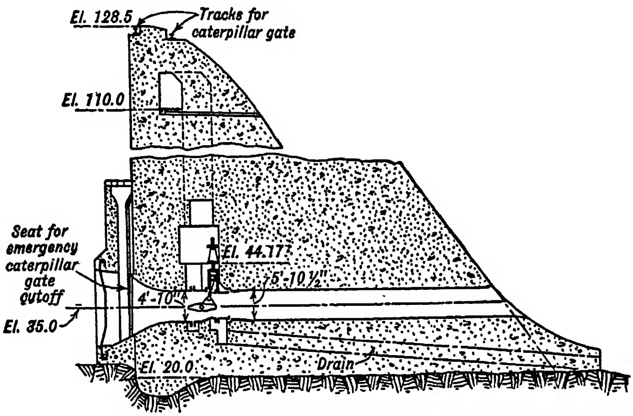


FIG. 39A. Sturgeon pool development. Cross-section through discharge valves, United Hudson Electric Corporation.

sluice is ordinarily operated and will then be above water surface most of the time. Trouble from freezing can be eliminated by providing at the outlet of the sluice a temporary cover which may be removed or washed out when the sluice control is opened.

The entrance to the sluice is usually protected by heavy rack bars, as shown in Fig. 39A, having a clear opening equal to about one-third the small diameter of the sluice control. However, some of the logs and trees which are stopped by the racks will pass part way through them and may interfere with the closure of the sluice if the racks are too close to the control. Therefore the racks should be located 20 to 30 ft above the control, depending upon the length of expected logs and trees. Racks are sometimes omitted, particularly for very deep pools.

Provision should be made for closing the upper end of the sluice to facilitate removal of the control for repairs. Stop logs are used for this purpose in low dams. For high dams, a seat is provided to receive a bulkhead or gate, which is lowered from the crest.

As the inside of the dam is very damp, geared hoists, especially if motor-controlled, require constant care to keep them in good condition. Oil-pressure cylinder hoists are most adaptable to sluice gates and valves inside the dam, as far as cost of maintenance is concerned.

Guard gates are frequently used in cases where it would be very difficult, on account of the great depth of water, to plug the upper end of the sluice for repairs to the control or lining. The guard gate, being operated very infrequently, is not likely to be damaged. Both slide gates and butterfly valves have been used in many installations for this purpose.

For a very high dam, it is advisable to install outlets at several different elevations so that they may be operated successively at relatively low head as the water is drawn down in the pond.

When the control is placed at the downstream end of the sluice, a watertight steel lining from headwater to the control is generally desirable. If a complete

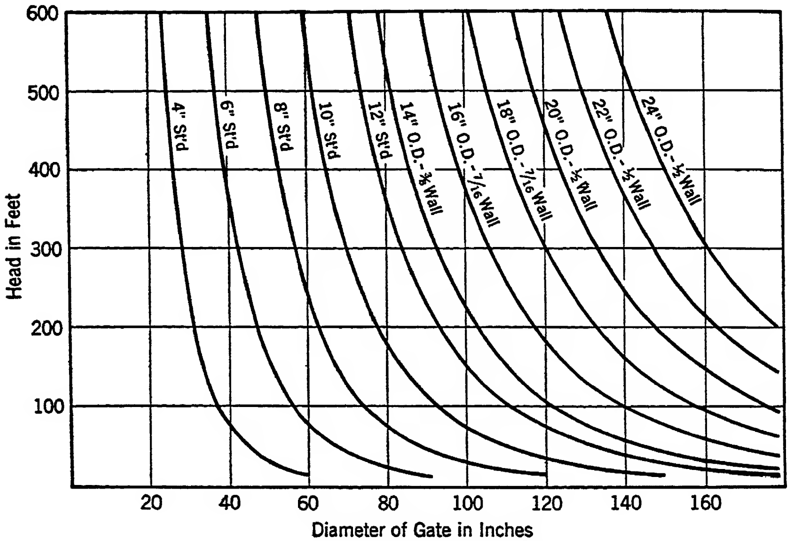


FIG. 40. Single air vent pipe sizes for ring follower and paradox gates (*"Dams and Control Works," by U. S. Dept. Interior, Bureau of Reclamation.*)

lining is not provided, full uplift of headwater should be considered as acting in the vicinity of the sluice.

(*b*) *Location of control.* The ideal location for control of discharge, from the hydraulic standpoint, would be at the outlet end of the sluice. The surplus energy of the discharge would then be expended outside of the structure, and the entire sluice would be subjected to positive pressure. Such an arrangement is frequently impracticable or uneconomical for structural or other reasons.

(*c*) *Ventilation.* When the control is within or at the upper end of the sluice, air must be introduced immediately below the control point to avoid negative pressures. Fig. 40 gives the sizes of air vents used by the U. S. Bureau of Reclamation for slide gates. Tests at Tygart Dam showed that slide valves 5 ft 8 in. by 10 ft 0 in. operating under a head of 90 ft draw at the rate of 166 cu ft of free air per sec under some circumstances.

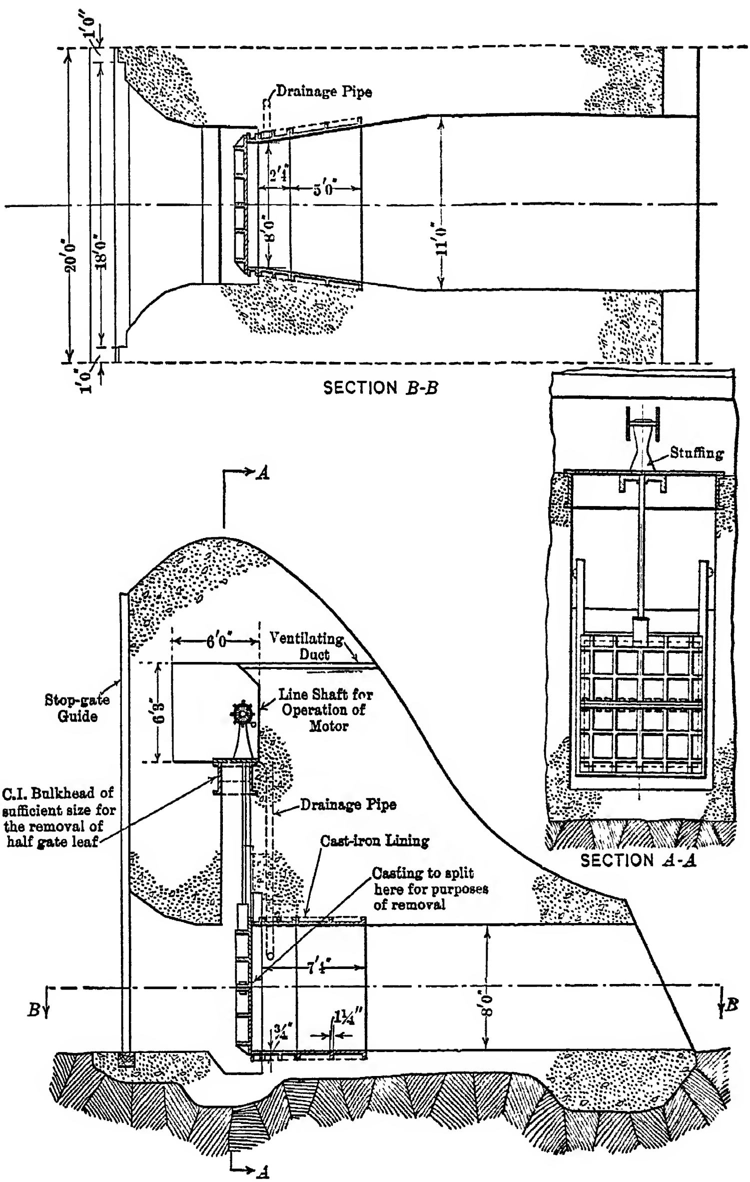


FIG. 41. Arrangement of 8 × 8-ft. sluice gates, Stevens Creek Development on Savannah River.

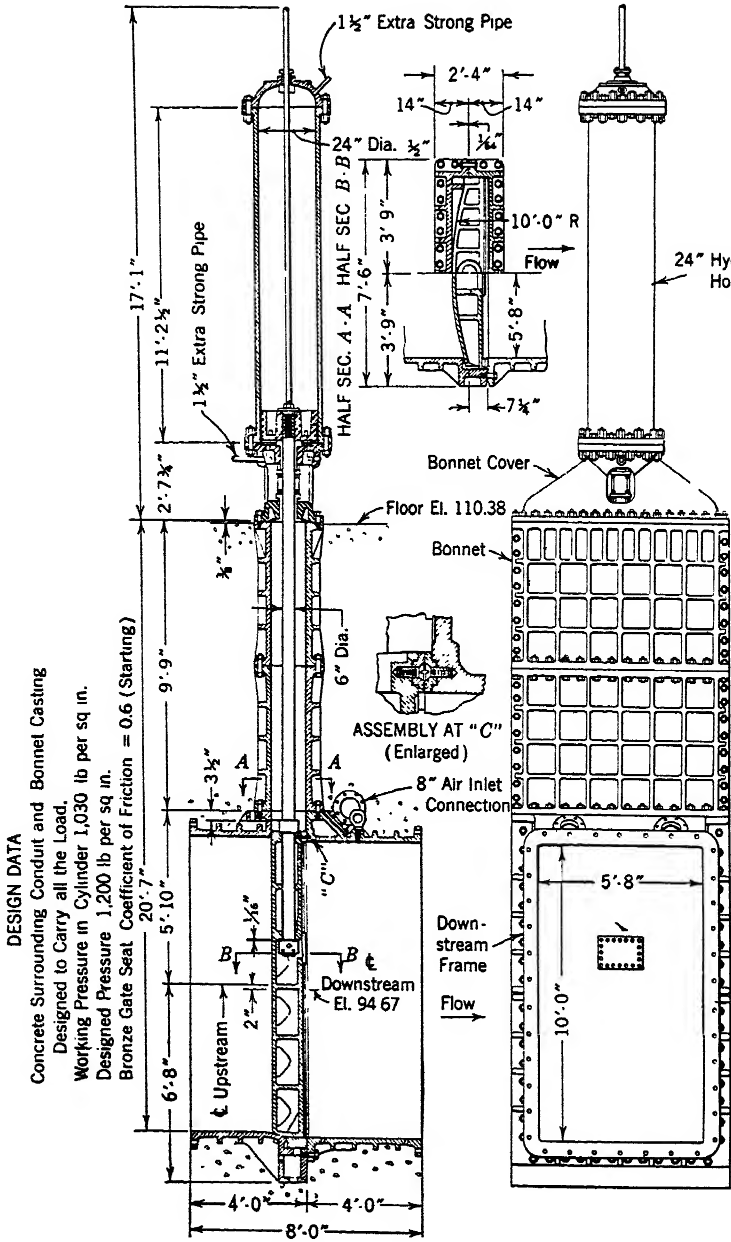


Fig. 42. U. S. Bureau of Reclamation slide gate.

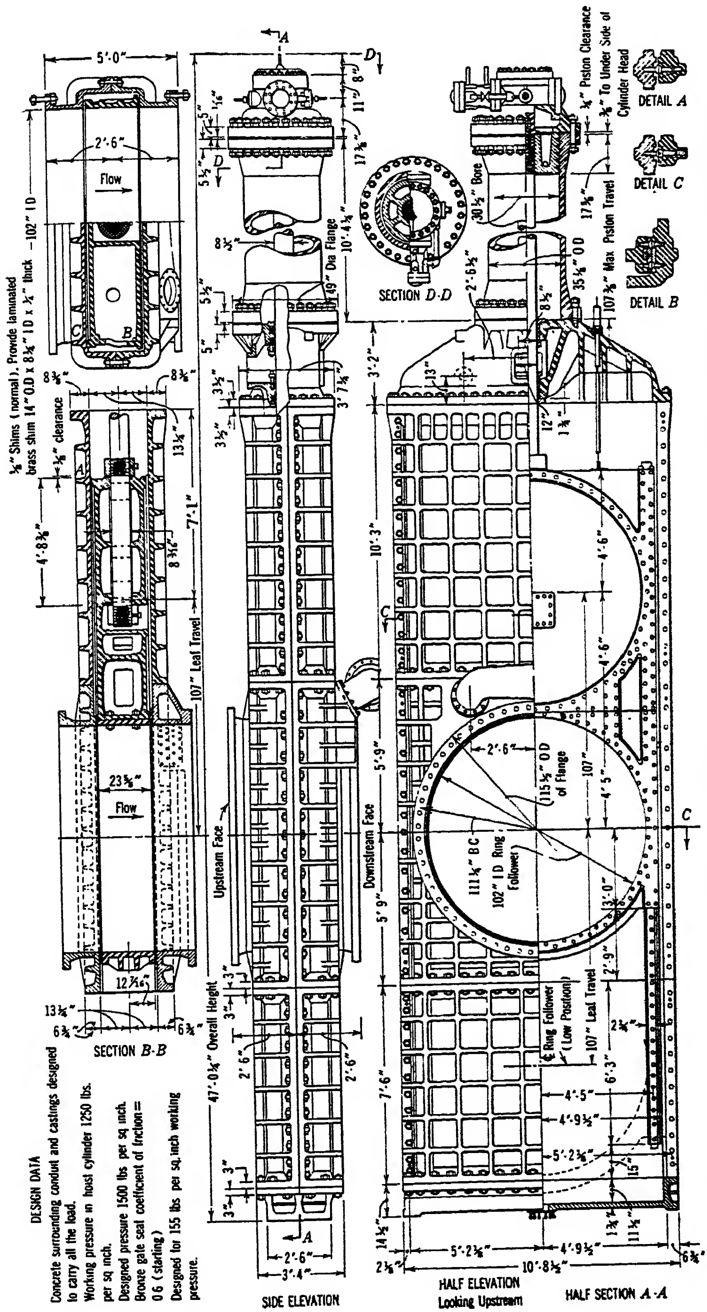


FIG. 43. Ring follower emergency gate. ("*Dams and Control Works*," by U. S. Dept. Interior, Bureau of Reclamation.)

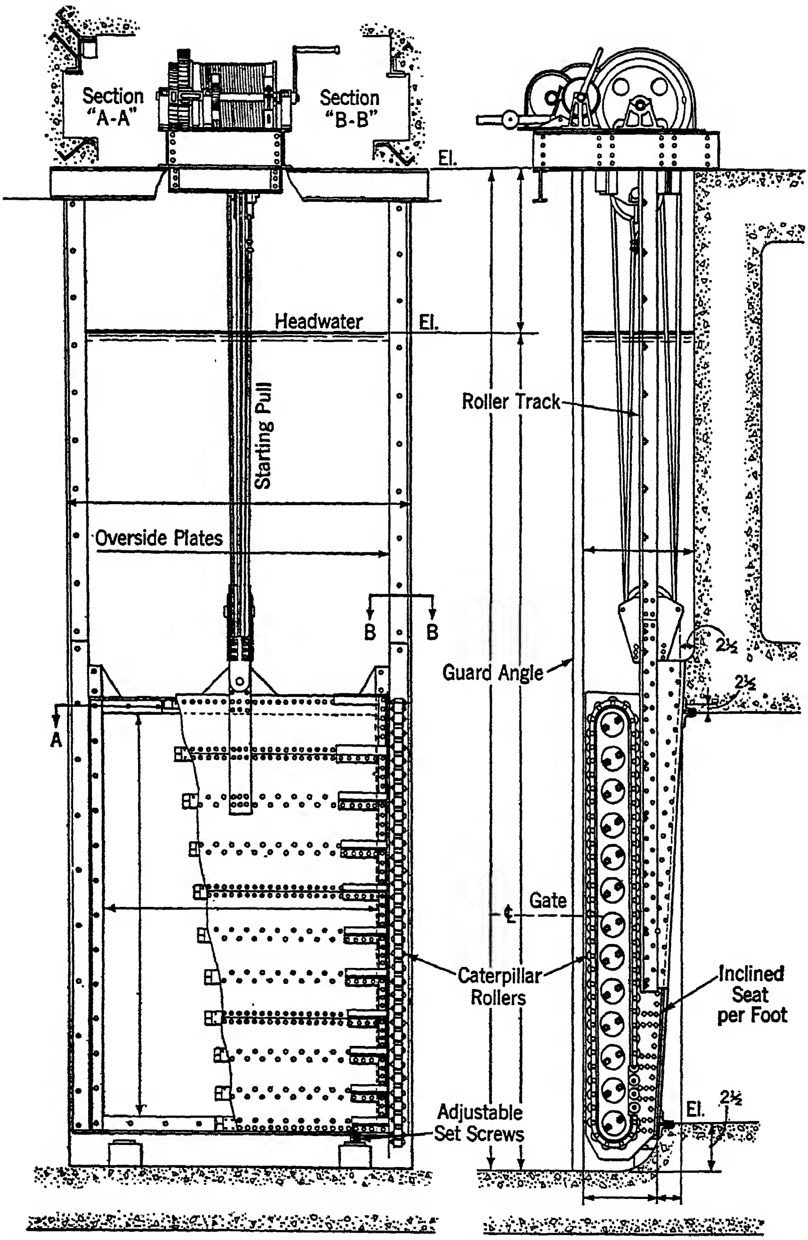


FIG. 44. Broome self-closing sluice gate. (*Catalogue No. 28, Philips & Davies, Inc. Kenton, Ohio.*)

(*d*) *Plain sliding gates.* A number of styles of plain sliding sluice gates are available commercially for controlling sluices of small area and operation under moderate heads. In selecting gates of this type, consideration should be given to loads imposed by dynamic effects (water hammer) as well as the static head. Fig. 41 shows a typical installation of this kind.

(*e*) *U. S. Bureau of Reclamation slide gate.* The U. S. Bureau of Reclamation has developed a design of slide gate that has been used successfully for heads up to 300 ft. The largest size controls an opening 5 ft 8 in. wide and 10 ft 0 in. high. The Bureau recommends this type of gate for regulating purposes for heads up to 70 ft. For greater heads, operation only in the fully opened or closed position is recommended.

A typical design for this type of gate is shown in Fig. 42. Frames and gates are made of cast iron for lower heads and cast steel for high heads. The gate bears upon bronze strips of special analysis which are mounted upon both the gate and the frame. The composition of the two contacting strips is slightly different. Gates have been constructed of bronze castings where corrosion from acid water was anticipated and on the Loyalhanna Dam, near Saltsburg, Pa., the fixed parts were built of welded stainless clad steel plate and constructed so that all exposed parts of the embedded structure were protected by stainless steel surfaces.

The embedded castings are substantially constructed but depend upon the support of the surrounding concrete to resist the water pressure.

Gates of this type are operated by hydraulic cylinders in line with the gate stem. The cylinders are generally made of forged steel and operate on oil at pressures between 500 and 1000 psi.

(*f*) *Ring follower gates.* The ring follower gate illustrated in Fig. 43 is essentially the same as the slide gate except that it is designed for operation in a sluice of circular cross-section. When fully opened, the follower ring closes the opening in the frame, making a continuous cylindrical water passage. Gates of this type are most frequently used as guard gates ahead of needle valves.

(*g*) *Tractor gates.* Tractor type gates differ essentially from plain slide gates in that the gate leaf travels upon rollers which are interposed between the bearing surfaces. Since the bearing friction is relatively low, the weight of the leaf may be used to supply the closing force and the gate may be operated by a simple tension member, such as a cable or chain from the top of the dam. For such a method of operation, the gate must be mounted upon the face of the dam or in a well in the mass of the dam. Gates of this type are generally used in masonry dams as guard gates in sluices that are normally controlled by pressure outlet valves.

Both caterpillar roller chains and fixed rollers have been used for the bearings of tractor type gates. The requirements for design are essentially the same as for roller-bearing crest gates described in Art. 5*c*.

Since gates which travel upon roller bearings do not make watertight contact on the bearing surfaces as do plain slide gates, some separate means of sealing is needed. Broome gates travel upon caterpillar roller chains but seat upon metal surfaces which are inclined relative to the path of travel, as in Fig. 44. The

U. S. Bureau of Reclamation has developed a tractor type gate in which a roller-bearing wedge is utilized to move the gate leaf off its seat as the first step in the

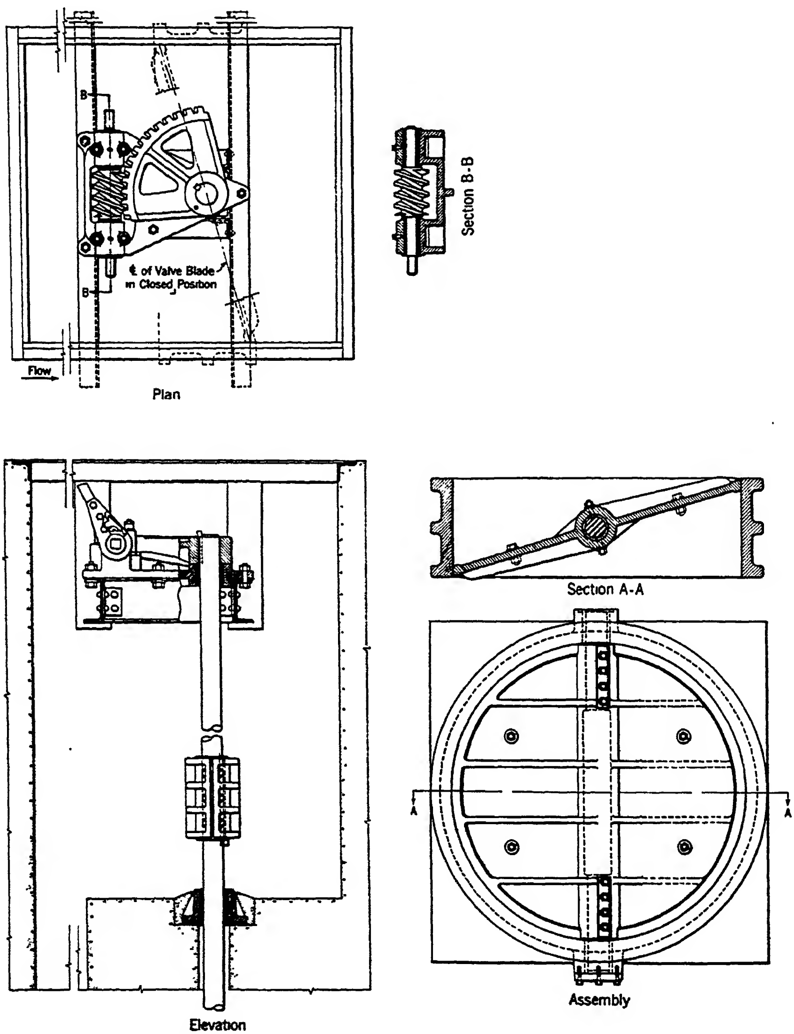


FIG. 46. Butterfly valve, Tygart Dam, Grafton, W. Va. (*U. S. Engineer Office, Pittsburgh, Pa.*)

raising operation. For side sealing surfaces that are parallel to the line of travel, flexible metallic or rubber seals have been used with low heads, and the U. S. Army Engineers have used solid rubber seals in sliding contact on tractor type gates designed for 160 ft of head.

(*h*) *Butterfly valves.* Circular butterfly valves have had considerable application, both as guard gates and as operating valves under moderate heads. This type of valve has the advantage of requiring relatively small space outside of the lines of the sluice. A large torque is required for operation because of the reaction of the flowing water, and the disk must be held in position firmly to prevent fluttering.

Fig. 46 shows a hand-operated valve of this type used in a drainage sluice for the cushion pool at Tygart Dam with a head of about 22 ft.

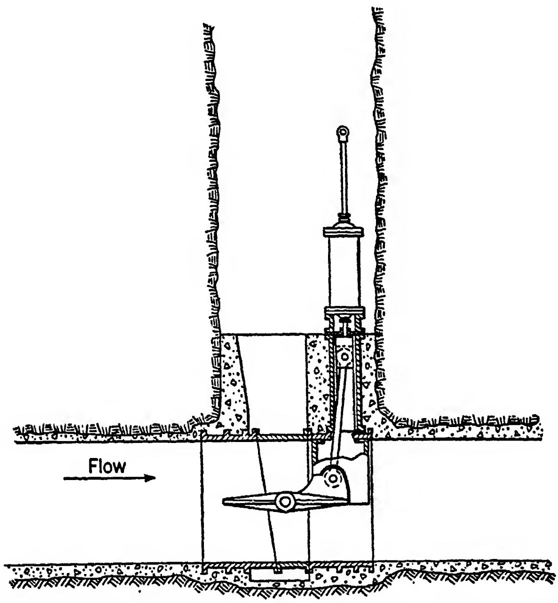


FIG. 47. Dow valve. (*From Bull. 113, June 1924, p. 6, by S. Morgan Smith Co., York, Pa., and Coffin Valve Co., Neponset, Mass.*)

In the Dow valve shown in Fig. 47, the operating force is applied through mechanical linkage rather than through torsion in the valve shaft as in other types.

Fig. 48 shows a butterfly valve developed by the U. S. Bureau of Reclamation for high heads. The disk is actuated by a hydraulically operated rotor and held in position by a hydraulically operated brake.

(*i*) *Radial gates.* Radial gates have been used mostly for low heads. They are essentially the same as taintor gates described in Art. 5*i*, except for the addition of a top seal. Radial gates arranged in a position that is reversed to the usual taintor gate arrangement, i.e., with the hinge on the headwater side, are used for lock valves under heads up to 70 ft. This arrangement places the open gate shaft on the upstream side of the closure, where it serves to absorb surge pressures.

(*j*) *Needle valves.* Needle valves are commonly used for regulating discharge with heads greater than 50 ft. The flow is discharged through an annular orifice, the area of which is controlled by an internal plunger. The plunger is actuated

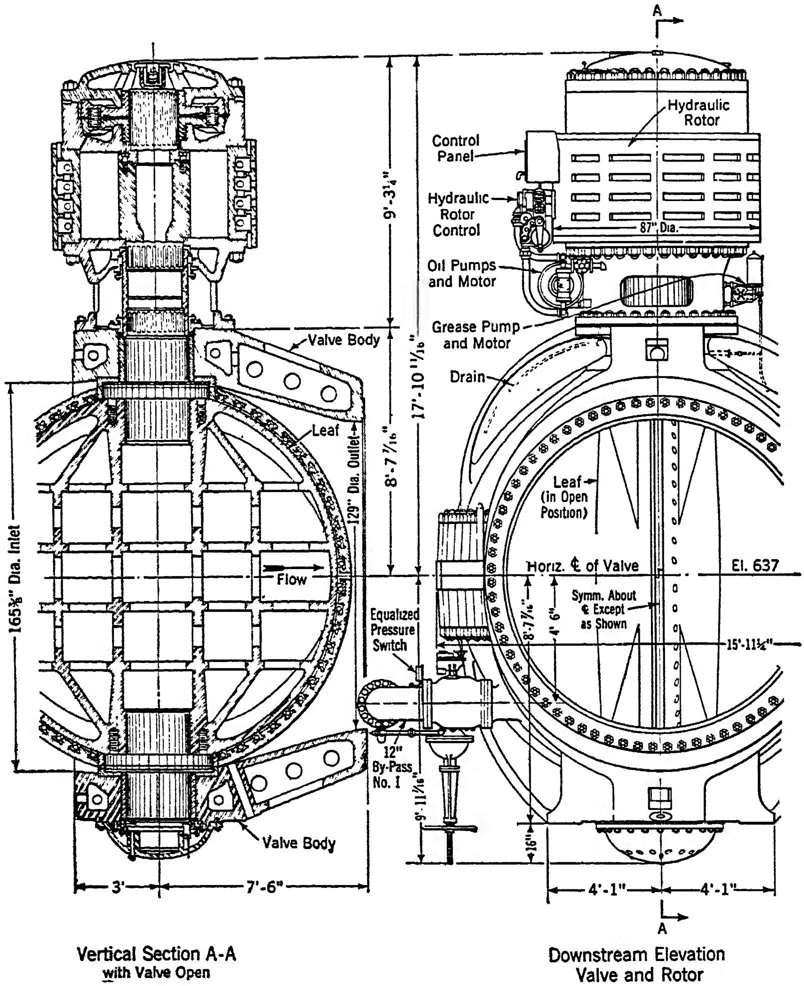


FIG. 48. Butterfly valve and rotor. ("*Dams and Control Works,*" *by U. S. Dept. Interior, Bureau of Reclamation.*)

by headwater pressure and is controlled by a device which automatically retains it in position by balancing the water pressures in certain interior chambers. Needle valves are usually, and preferably, installed to discharge against atmospheric pressure without conduits beyond the outlet end. It is sometimes desirable or necessary to install a valve of this type in an intermediate position in the sluice. In this case, an expanding conduit section is installed below the outlet

and air is frequently admitted from an annular chamber surrounding the conduit close to the outlet of the valve. Needle valves may be damaged from freezing when exposed to low atmospheric temperatures, and their working parts require some maintenance attention. For these reasons, guard gates are needed in the conduit and provisions must be made for draining the valve body and interior chambers.

(*k*) *Larner-Johnson valve.* An example of the Larner-Johnson valve is shown in Fig. 49. The flow passes through the outer annular passage. In operation,

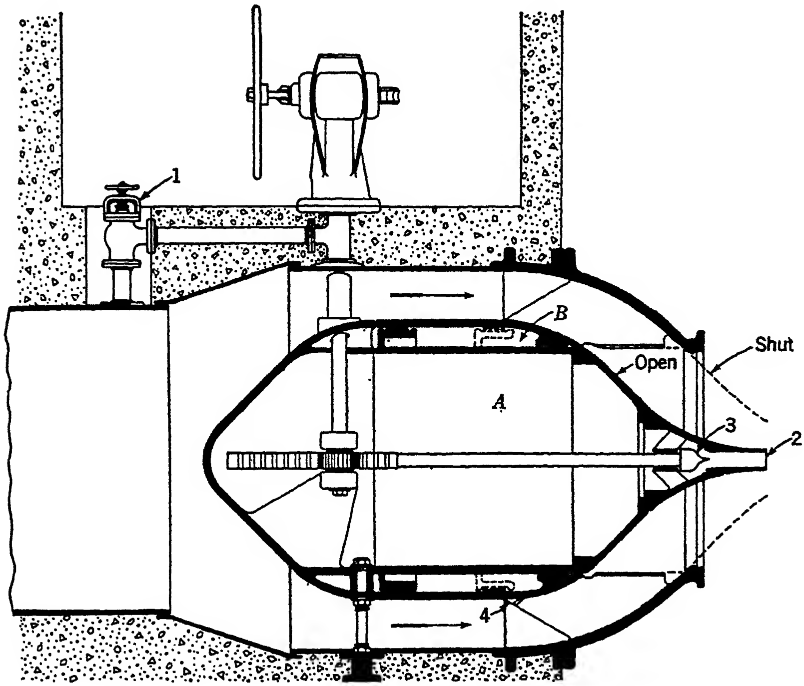


FIG. 49. Larner-Johnson Valve. (*Bull. 61, March 1936, I. P. Morris Division, Baldwin-Southwark Corporation, Philadelphia, Pa.*)

Chambers A and B are connected to headwater. A pilot valve (3) in the head of the plunger permits the release of water from Chamber A and is actuated by the hand control. When the pilot valve is closed enough to restrict the flow through the nozzle at the end of the plunger, the pressure in Chamber A is raised and the plunger is advanced. Advance of the plunger tends to open the pilot valve, thus stabilizing the position of the plunger in accordance with the setting of the manual control. Similarly, opening of the pilot valve decreases the pressure in Chamber A and retracts the plunger.

(*l*) *Internal differential needle valves.* The internal differential needle valve, as developed by the U. S. Bureau of Reclamation, is shown in Fig. 50. The distinguishing feature of this type of valve is an external control valve, known as

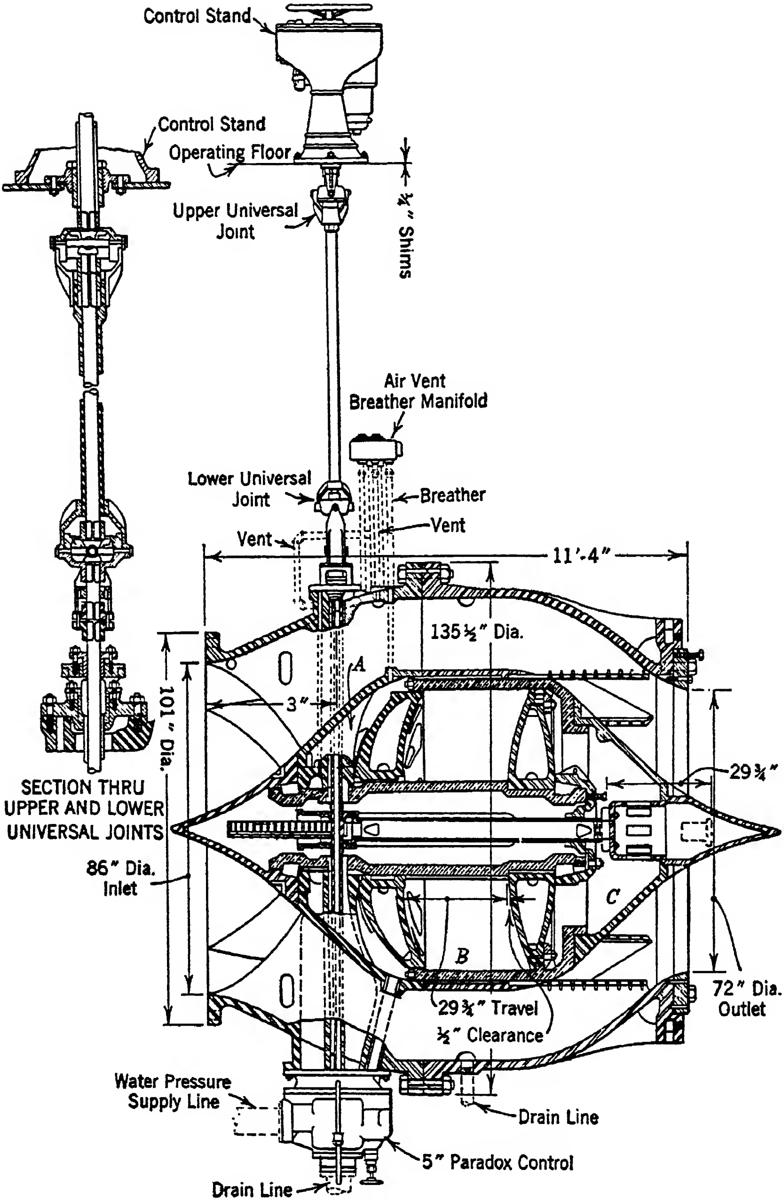


FIG. 50. Needle valve ("*Dams and Control Works,*" *by U. S. Dept Interior, Bureau of Reclamation.*)

the "Paradox Control," which controls the pressures in the interior chambers. The needle valve is divided into three chambers designated as A, B, and C. Chamber A is drained to atmospheric pressure. To close, conduit pressure is

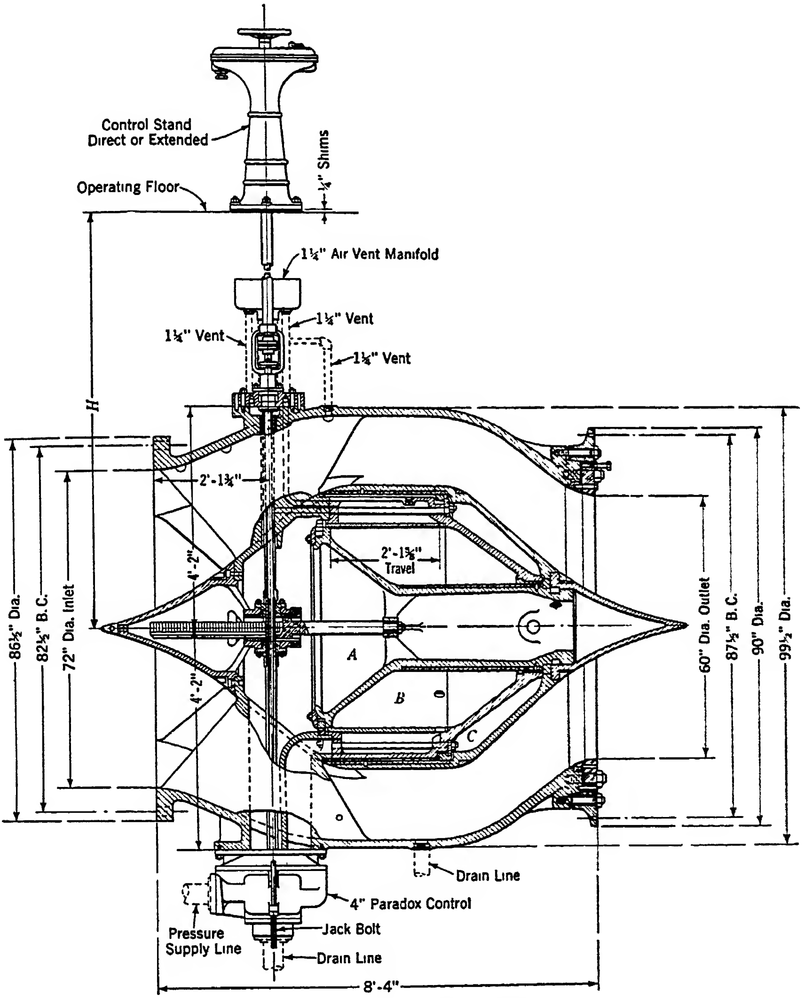


FIG. 51. Interior differential needle valve. (*"Dams and Control Works," by U. S. Dept. Interior, Bureau of Reclamation.*)

admitted to Chamber C, and Chamber B is drained to atmospheric. To open, these pressures are reversed. The paradox control is a piston valve arranged on the "follow-up valve" principle, i.e., the mechanism is arranged so that, with any manual setting, movement of the plunger in either direction will readjust the control port openings, so as to arrest or reverse the plunger movement. The

plunger is in this manner kept close to the position determined by the manual setting of the control. When the manual control is moved, the plunger moves in the corresponding direction until its position is again stabilized by the readjustment of the control valve by the "follow-up" mechanism. The Bureau of Reclamation recommends for a conservative value 0.725 for the coefficient of dis-

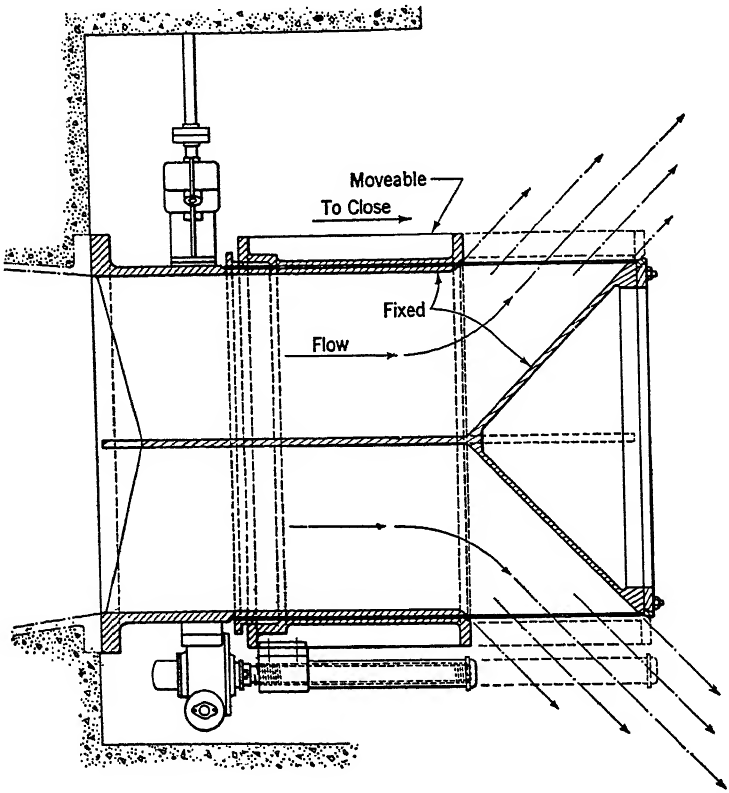


FIG. 52. Howell-Bunger valve. (*Courtesy S. Morgan Smith Co., York, Pa.*)

charge of this type of valve based on the "nominal" diameter, i.e., the diameter of the outlet flange opening. However, this value is approximate, since the details of the valve are capable of alteration with resulting alteration in the coefficient of discharge.

(*m*) *Interior differential needle valves.* The interior differential needle valve is a more recent development of the needle valve by the U. S. Bureau of Reclamation. It differs from the internal differential needle valve in the arrangement of the internal chambers, as shown in Fig. 51. The operation is essentially the same except that Chamber A is made a pressure chamber and acts with Chamber C to close the valve.

(*n*) *Howell-Bunger valve.* The Howell-Bunger valve, shown in Fig. 52, is designed for use only at the outlet end of the conduit. In this type of valve the jet is divergent instead of convergent as in the needle valve. Therefore its jet is dissipated over a very large area with resulting reduction in scour below the outlet.

It consists of a "fixed" cylindrical body with a cone-shaped lower end causing a flaring discharge which can be closed by the "movable" cylinder.

The manufacturer recommends a conservative coefficient of discharge of 0.90.

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INDEX